Underground Disposal of Storm Water Runoff

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Welcome to FHWA-TS-80-218-Underground Disposal of Storm Water Runoff

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Background

Artificial replenishment of groundwaters by surface infiltration has been practiced for many years. As early as 1895, flood waters of San Antonio Creek in Southern California were conserved by spreading them on the alluvial fan at the mouth of San Antonio Canyon. After the construction of the City of Fresno's sewerage system in 1891 and until 1907, the city disposed of all of its wastewater on a 40-acre (161,880 m^2) tract. Over the years, Fresno has increased the size of its "Sewer Farm", which uses some surface sprinkling and a large number of infiltration ponds, covering some 1,440 acres (5.8×10^6 m^2) of land in 1972. Although some storm water reaches the site, most of the flows are treated sewage effluent.

Richter and Chun in 1961(1) reported that fifty-four agencies were actively practicing artificial ground water recharge in California, alone, in 1958. Many agencies elsewhere artificially replenish groundwaters. Barksdale and Debuchananne in 1946(2) describe the practice in New Jersey; Boswell in 1954 (3) discusses artificial replenishment of groundwater in the London Basin; Brashears in 1946(4) provides information on artificial recharge as practiced on Long Island, New York; Cederstrom and Trainer (5) presented information in 1954 about groundwater recharge in Anchorage, Alaska; Kent(6) reported in 1954 on practices in the Union of South Africa; methods used in southwest Africa were described by Martin in 1954 (7), and Sundstrom and Hood in 1952(8) describe the results of artificial recharge of groundwater at El Paso, Texas. An annotated bibliography on artificial recharge of groundwater through 1954 is presented in the U.S. Department of the Interior, Geological Survey, Water Supply Paper 1477(9). For those wishing to review the subject in detail, other published reports are available.

The U.S. Department of Agriculture, in 1970 (10), published a summary of the principles of groundwater recharge hydrology which described the more common methods used. These include: basins, ditches or furrows, flooding, natural stream channels, pits and shafts, and injection wells. In the research report, "Infiltration Drainage of Highway Surface Water" (1969), Smith, et al(11) give a summary of the principles of infiltration drainage for highway surface water, and descriptions of the various kinds of systems with numerous references.

During the development of this manual, questionnaires were sent to a number of agencies and engineering consulting firms for the purpose of ascertaining to what extent infiltration systems were being utilized throughout the nation. The results of these inquiries are presented in Appendix A. Although these results represent only a sampling, they seem to indicate extensive utilization of infiltration drainage in localized areas of the country. In other areas, experience with infiltration procedures is almost nonexistent. Environmental and legal restraints are frequently cited as factors prohibiting the use of these systems. These restraints are addressed in Chapter 3-A of this manual.

The following sections provide additional state-of-the-art information dealing with facilities constructed for subsurface disposal of storm water. These systems can provide for water conservation by groundwater replenishment and/or prevention of salt-water intrusion; or for disposal of storm water runoff. Basins, trenches, and infiltration well systems are discussed.

The final section of this chapter, "New Products and Methods for Aiding Infiltration", describes recent developments that have been beneficial to the planned infiltration of storm water.

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1. Infiltration Basins

Infiltration basins are of natural or excavated open depressions of varying size in the ground surface for storage and infiltration of storm water. Weaver in 1971 (12) presented theoretical and experimental work done by the New York State Department of Transportation to develop a procedure for designing infiltration basins. Weaver points out that increasing demands for fresh water and dwindling supplies, together with the advantage of constructing short trunk sewers leading to basins rather than the longer sewers that would have been needed, motivated the use of the infiltration basins on Long Island. More than 2000 infiltration basins are now in use on Long Island, New York.

In a discussion of artificial recharge in water resources management, Dvoracek and Peterson, in 1971 (13), point out that maintenance requirements of infiltration basins are usually minimal. They state that, "cleaning the sediments from pits, trenches, and spreading basins is a relatively simple operation, possibly involving nothing more than tillage of these areas. In extreme cases, physical removal of sediment may be necessary." One method to partially offset the need for maintenance in areas of extreme climatic change is to allow the facilities to experience freeze/thaw action. Pit recharge rates have been known to increase sixfold due to freeze/thaw conditions during winter months. A physical breakdown of the surface seal seems to occur. facilitating self-maintenance.

Infiltration basins have been used extensively for many years in California's San Joaquin Valley in areas where immediate discharge of storm water from roadway rights-of-way would normally overtax the adjacent surface drainage systems or where an outfall is not available (11). They serve as storm water retention basins with possible infiltration benefits. However, infiltration is generally a secondary benefit, due to the low permeability of the clayey soils that exist throughout the San Joaquin Valley. In most cases it is considered a safety factor in designing the necessary storage volume-of the systems. Other similar experiences are presented in Appendix A.

Many cities and local park districts combine plans for infiltration basin construction with green-belt zoning. This multi-use merging of the two facilities permits development that is both practical and aesthetically pleasing. An example of a typical detention-infiltration basin in a city park is shown in Figure 2-1. Details on the design and construction of these basins can be found in subsequent chapters of this manual. The American Public Works Association Special Report No. 43(14) is also an excellent reference for the location and design of detention systems in urban areas.

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2. Infiltration Trenches

Infiltration trenches may be either unsupported open cuts with side slopes, flattened sufficient for stability; or essentially vertical-sided trenches with concrete slab cover, void of both backfill and drainage conduits where side support is not necessary (Figure 2-2); or trenches backfilled with coarse aggregate and perforated pipes where side support is necessary (Figure 2-3). Dvoracek and Peterson in 1971 (13) describe the use of unsupported open recharge trenches as an alternative to pit recharge. "A long narrow trench, with its bottom width less than its depth . . . is utilized rather than the large rectangular pit. Dependent upon the infiltration characteristics of the material into which the trench penetrates and the location of the water table, high rates of recharge are generally expected". Infiltration trenches have been used successfully in Southern Florida under high groundwater conditions but have required special engineering considerations. The infiltration trench is a modification of the infiltration basin, discussed in Section 1. Porter in 1976 (15) discusses the advantages of covered drainage trenches for "recharge to ground" of storm water runoff. A typical trench cross section is shown in Figure 2-4.

Figure 2-1. Typical Detention-Infiltration Basin in Green Belt-Area *(Courtesy of CALTRANS)*

Figure 2-2. Infiltration Trench with Stable Vertical Side Walls in Native Material with Concrete Slab Cover (Miami Area) ***(Courtesy of Bristol, Childs & Associates, Coral Gables Florida)***

The addition of perforated pipe to the infiltration basin concept increases the exfiltration from the trench by more than 100 times that of conventional "French drains" or dry wells which are limited by cross-sectional area. It also serves the function of collecting sediment before it can enter the coarse rock backfill. As collected, sediments are distributed throughout the length of the freeflow area, and clogging is minimized. For example, the sediment-laden water must flow through the cross-section of the conventional French drain to flood the trench and gain access to the trench wall. The perforated pipe distributes the water immediately for its full length, providing immediate access to the trench wall. A French drain 8 ft (2.44 m) deep and 4 ft (1.22 m) wide must exfiltrate through a 32 ft^2 (2.98 m^2) cross-sectional surface. An infiltration trench with 36 inch (0.915 m) diameter pipe running between inlet structures 200 ft (61 m) apart exfiltrates to the coarse backfill rock for the full trench length through an area of πd x R = 3.1416 x 3 ft (0.915 m) x 200 ft (61 m) = 1,885 ft^2 (175.3 m^2).

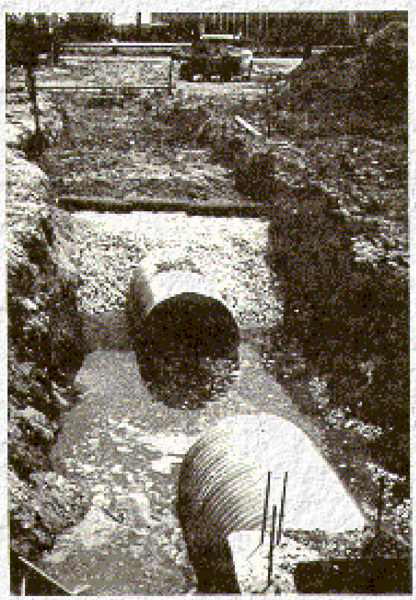

Figure 2-3. Infiltration Trench with Perforated Pipe and Coarse Rock Backfill. Note Groundwater Level in Excavation (Miami Area) ***(Courtesy of Dade County Dept. of Public Works, Miami, Florida)***

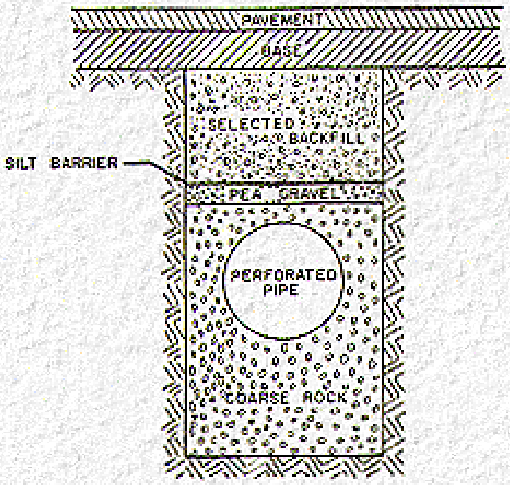

Figure 2-4. Typical Cross-Section of Infiltration Trench *(Courtesy of Dade County Dept. of Public Works, Miami, Florida)*

Infiltration trenches have been used extensively in Dade County, Florida, and in other areas of the State, as well as in some parts of Canada, as discussed by Porter (15) and Theil (16). A listing of performance information on various installations is provided in Appendix B. Refer to Section 2-4 of this chapter, "New Products and Methods for Aiding Infiltration" for a description of perforated pipe. Examples of these systems are also described and illustrated in detail in other chapters of this manual.

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3. Infiltration Wells

Recharge or infiltration wells have been used for many decades for conducting water into the ground. Perhaps the oldest kind is the "dry well", which is a small-diameter hole or pit dug into the ground for the disposal of water that has no natural drainage. A dry well is usually filled with pea gravel, coarse sand, or other aggregate; or contains a slotted or perforated pipe, backfilled with materials which allow water to penetrate and soak into the ground, while preventing collapse of the walls. Frequently, a layer of filter sand is placed in the top few inches (0.1 m±) of a well and mounded up slightly over the well, to trap silt and other sediment that might clog the well. The sand can be periodically, removed and cleaned, or replaced. An enlarged version of the dry well is the "seepage pit" used for disposal of sewage from septic tanks. These are discussed in detail by the U.S. Department of Health, Education, and Welfare's Public Health Service Publication No. 526 (17). In some States, seepage pits are permitted when absorption fields are impracticable, and/or where the top 3 or 4 feet (0.9 or 1.2 m) of soil is underlain with porous sand or fine gravel and the subsurface conditions are otherwise suitable for pit installations.

Abandoned wells, or wells specifically designed for artificial recharge, have been used for many years to inject water into the ground. The U.S. Department of Agriculture Publication 1970 (10), states: "The use of injection wells is confined largely to areas where surface spreading is not feasible because extensive and thick impermeable clay layers overlie the principal waterbearing deposits. They may also be economically used in metropolitan areas where land values are too high to use the more common basin, flooding, and ditch-and-furrow methods."

This publication also points out: "Many attempts to recharge groundwater through injection wells have been disappointing. Difficulties in maintaining adequate recharge rates have been attributed to silting, bacterial and algae growths, air entrainment, rearrangement of soil particles, and flocculation caused by reaction of high-sodium water with soil particles."

Cased, gravel-packed wells have been used for injecting good quality water to provide a barrier to salt water intrusion. Bruington and Seares (18) in 1965 reported "The control of intrusion of coastal groundwater basins by sea water has become of economic importance in groundwater basin management."

Many researchers have contributed to the body of knowledge on flows to and from wells. Muskat (19) in 1937 developed theories for steady-state seepage toward a single well, small groups of wells, and infinite sets of wells in one-, two-, and three-line arrays. His work provides the background for many refinements in seepage theory that have been developed in recent years. Hantush (20) (1963), Glover (21) (1966), Leonards (22) (1962), Peterson (23)

(1961), Harr (24) (1962), and Todd (25) (1959) are just a few references on well theory.

Kashef (26) in 1976 reported the results of a theoretical study of the effect of injection into batteries of wells on salt-water intrusion. His report presents charts that may be useful to those managing salt-water intrusion systems using injection wells.

Even though well theories can be useful to those designing water injection or recharge wells, numerous practical considerations ultimately determine their effectiveness. For example, Reference (10) from the U.S. Department of Agriculture contains the following statement, ". . . the Los Angeles County Flood Control District in California has successfully operated injection wells as part of a largescale field experiment to ascertain the feasibility of creating and maintaining a fresh-water ridge to halt seawater intrusion in the Manhattan-Redondo Beach area in Los Angeles County. In general, it has found that gravel-packed wells operate more efficiently and require less maintenance than non gravel-packed wells. At Manhattan Beach, California, a 24-inch (0.61 meter) gravel-packed well with an 8-inch (0.203 my casing was found more desirable for recharging purposes. On Long Island, New York, where cooling water is returned to the ground-water basin, a minimum casing size of 8-inches (0.203 m) and a minimum packing of 2-inches (0.05 m) have been recommended."

The Transportation Laboratory of the California Department of Transportation in a 1969 report(11) discussed recharge or "drainage" wells as follows: "Drainage wells are basically water supply wells operating in reverse, although, in practice, they have many unique features and problems. There are also several types, ranging from simple gravel-filled shafts to highly sophisticated pump injection wells. Like basins, they have both good and bad features. Wells require a minimum of space and may be designed with very little unsightly surface structure. They can be extended through impervious soils down to permeable sand or gravel, and will drain a small area fairly rapidly when surface runoff is of satisfactory quality.

"Unfortunately, wells clog up very easily when the water contains silt or sediment, and cleaning or restoration can be difficult. Drainage wells are readily capable of polluting groundwater supplies and health departments have strict regulations regarding them. Capacity for drainage is difficult to predict: one well may have a good rate of infiltration, while another 50 feet (15.3 m) away will drain very poorly. The cost of well construction and maintenance makes well drainage a fairly expensive method of disposal. Basins are much more economical in terms of cost per unit volume of water drained. Normally, a drain well should be considered for disposal of small quantities of water, or as a supplement to recharge basins or some other type of disposal system."

The City of Modesto, California, with an average annual rainfall of 12 inches (305 mm), makes extensive use of drain or rock wells to serve seventy percent of the city area. Their experience with over 6,500 individual installations has varied. Some wells, considered as marginal, have resulted in pending on streets following severe rainstorms. These facilities have required continuing maintenance. Figure 2-5 shows a typical cross-section of the standard "rock well" used in the City of Modesto for street drainage.

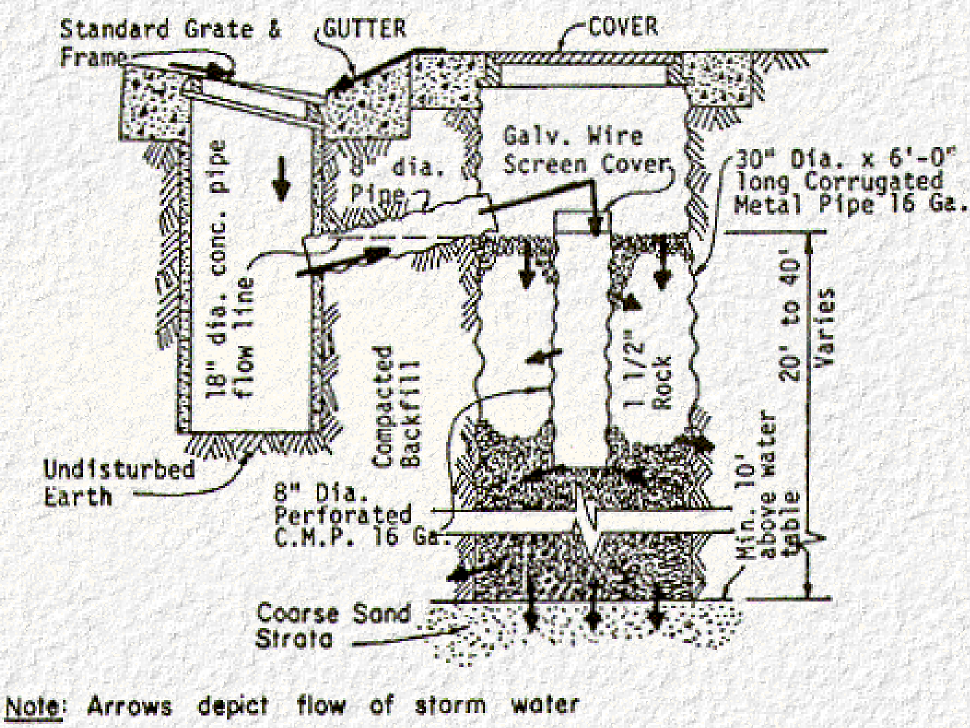

Figure 2-5. Cross-Section of Standard Rock Well (Drain well) Installation for Street Drainage *(Courtesy of Modesto, Calf. Dept. of Public Works)*

Infiltration wells or "diffusion" wells, as used by the New York Department of Transportation on Long Island are large, often very deep, concrete-lined pits. Weaver (12) states: "As used by this Department on Long Island, these have customarily been large vertical shafts constructed of reinforced concrete precast sections. The sections are 6-feet (1.8 m) high with a 16-inch (0.406 m) wall and an inner diameter of 10 to 16 feet (3.1 to 4.9 m). A diffusion (infiltration) well is constructed in the same manner as a drop shaft or open caisson. The shell sinks under its own weight as the soil at the bottom is excavated, and additional sections are added from the surface. By means of rectangular openings through the wall, each 10-foot (3.1 m) inside diameter section provides approximately 9.1 square feet (0.85 m^2) of effective lateral drainage area. When the shaft is completed, a heavy reinforced concrete cover is -placed over the top. The cover contains an open grating about 8 square feet (0.74 m^2) in size. Over the cover at the floor of the basin, a graded filter is placed to prevent silt from entering the well." Weaver points out that most of these wells have been carried at least 6 feet (1.83 m) below the water Table and often to depths between 100 and 200 feet (30.5 to 61 m). He indicates these shafts or wells have most often been used as a remedial measure to correct the results of inadequate design and/or inadequate maintenance of existing infiltration basins. Because of their high cost, there is a question as to whether this type of recharge well is justified on the basis of hydraulic conductivity. Weaver emphasizes that his department makes use of seepage analyses methods to estimate their inflow capacities even though the "design of a diffusion well is a multi-component, highly complex task." He also states: "Owing to their high cost and low efficiency, they are the least desirable method of disposing of highway drainage. Also, because of their low efficiency, a rather large infiltration basin is necessary merely to hold the storm inflow for eventual disposal by the diffusion well, so that wells are <u>not alternates</u> to basins they are an <u>extra cost</u> added to the basin cost."

Various patented dry well systems are available for subsurface disposal of stormwater. These systems are very similar to those previously discussed.

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4. New Products and Methods for Aiding Infiltration

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a. Synthetic Filter Fabrics

Many kinds of engineering and agricultural drainage systems make use of graded filters or multiple-layer drains for the safe removal of water from soil formations. When aggregates are used, their gradations are usually established with the well-known "Terzaghi" or "Bertram" filter criteria. These are discussed in Chapter 4-C, "Design of Storm Water Collection and Disposal Systems." Good quality mineral aggregates are virtually indestructible, and until recently have been economical and available in many geographical locations. However, as the supplies of dependable aggregates has diminished and the cost of placing more than one kind of aggregate (in trenches, for example) has increased, there has been an impetus to make use of the synthetic fabrics either to act as separators to keep fine erodible soils out of porous drains, or to work as filters to allow free flow of water while preventing the movement of the erodible soils. Barrett(27) (1966), Calhoun(28) (1972), Dunham and Barrett(29) (1974), the U.S. Army(30) (1975), Carroll(31,32) (1975, 1976), Rosen and Marks(33) (1975), Seemel(34) (1976), and many others worked with fabrics and developed standards and specifications for their use.

Polyvinylidene chloride, polypropylene, and other synthetic resins used in making filter fabrics are inert materials not subject to rot, mildew, or insect and rodent attack. They are, however, very sensitive to long term exposure to ultraviolet components of sunlight. Also, some are affected by alkalis, acidic material, components of asphalt, or fuel oils. If a fabric is substituted for an aggregate filter, care should be taken to prevent tearing or puncture of the fabric. Adjacent sheets should be overlapped and secured to prevent openings from developing.

To insure the required performance for the life expectancy of the project, synthetic fabrics (either woven or nonwoven) for infiltration systems or any other long-term application, must be carefully selected, based on the properties required. As with aggregate filters, fabric filters must provide two very important functions; (1) they must be able to prevent clogging of the drain by erodible soil or other material, which could also result in erosion, piping, or other problems with the facility being protected; and (2), they must not inhibit the free flow of water. In situations where the fabrics work only as separators, and there is no significant flow of water, they need only satisfy the first requirement.

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b. Precast Concrete or Formed-in-Place Perforated Slabs

The current emphasis on storm water management has resulted in new drainage concepts aimed at reducing the flow of storm water from developed areas. Smith(35) in 1974 described the use of porous precast paving slabs with perforations as a means to induce water to soak in and not flow off large parking areas, while these areas support grass in keeping with the "green belt" concept. This concept involves the use of proprietary forcers and patented processes to produce reinforced concrete with holes that allow water to soak in and grass to grow. These materials produce grassy looking parking areas that are self draining, mud-free, and attractive in appearance. In essence they produce a load-bearing lawn which can absorb a good deal of rain thereby reducing surface runoff.

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c. Porous Pavements

Porous pavements have been suggested in recent years to recharge groundwater supplies and reduce storm water runoff(36,37,38). These pavements allow storm water to infiltrate through the pavement surface and be stored in the structural section for eventual percolation through the underlying native soil. This idea may have merit for parking lots but is not recommended for pavements that are subjected to large numbers of repetitions of heavy wheel loads which could increase replacement and maintenance costs.

Porous pavements are designed based on the load-bearing capacity of a saturated subgrade for an expected number of wheel load repetitions. The porous structural section is designed with sufficient reservoir capacity to handle the design rainfall. To function properly and provide vertical drainage, the native subgrade soil should have high permeability. Figure 2-6 illustrates a structural section for a typical porous asphalt concrete parking lot pavement. The pavement provides storage for 4.20 inches (107 mm) of rainfall assuming 15 percent voids in the surfacing and 30 percent voids in the aggregate base.

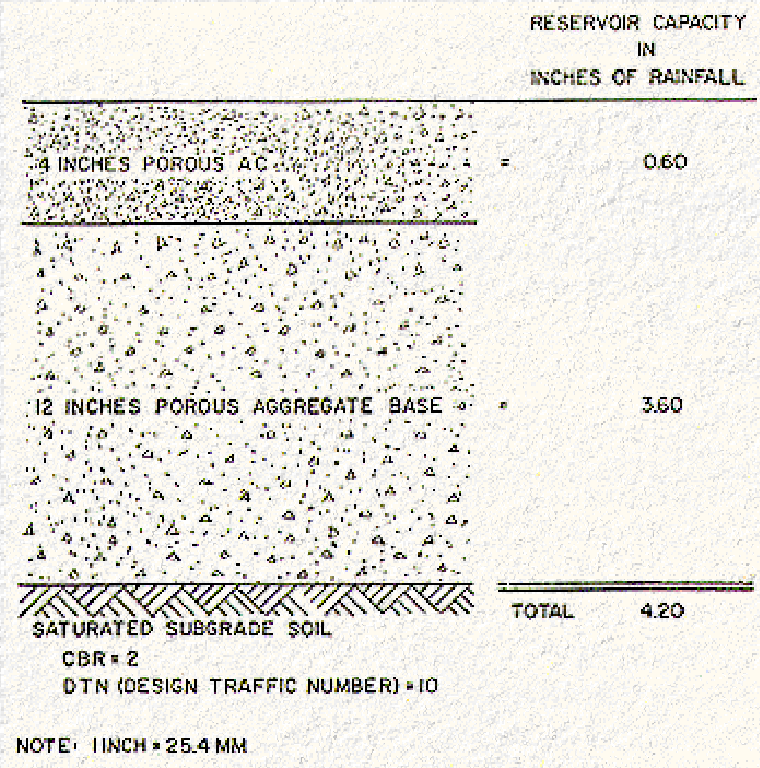

Figure 2-6. Typical Porous Asphalt Concrete Parking Lot Pavement *[After (36)]*

For design of pavements refer to the Design Manuals of the Asphalt Institute, Cement and Concrete Association or other references on the subject.

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d. Perforated or Slotted Pipe

The Corrugated Steel Pipe Institute "Drainage Technology Newsletter", November, 1976(15), describes a new type of fully perforated pipe for use in trench drains of the kind used by Dade County, Florida, for temporary storage and subsurface disposal of storm water. Pipes manufactured of aluminum, concrete, and other materials are also available for this application.

For perforated corrugated metal pipes [CMP 3/8 inch (9.5 mm)] diameter perforations uniformly spaced around the full periphery of a pipe are desirable. Not less than 30 perforations per square foot (0.093 m^2) of pipe surface should be provided. Perforations not less than 5/16 inch (8.0 mm) in diameter or slots can be used if they provide an opening area not less than 3.31 square inches (2135 mm^2) per square foot (0.093 m^2) of pipe surface. The photo in Figure 2-7 shows the inside of a metal pipe with perforations around the full periphery.

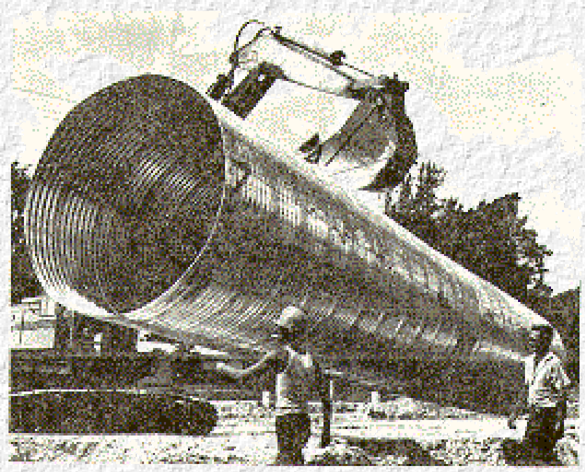

Figure 2-7. Typical Perforated Pipe for Infiltration Trench Construction *(Courtesy of Syracuse Tank & Manufacturing Co., West Palm Beach, Florida)*

The liberal number of holes are to insure free and rapid flow in and out of the pipe. The purpose of the large-sized pipes is to add to the total storage volume for storm water and to reduce the quantity of expensive rock backfill.

Coarse gravel or other aggregate is used for backfilling the trench around, below, and above the pipe so that part of the storm water is temporarily stored in the voids of the backfill. The photo in Figure 2-8 shows the typical coarse rock used for infiltration trench backfill. Experiments made by the Dade County Department of Public Works have indicated that 3/4 inch x 1 1/2 inch (19 mm x 38 mm) coarse gravel backfill with pipe systems having 3.31 square inches per square foot (23 x 103 mm^2/m^2) of perforations will provide pipe exfiltration rates which exceed the best infiltration rates of soils normally encountered in the field. Refer to Appendix C for information on experimental development tests by Dade County.

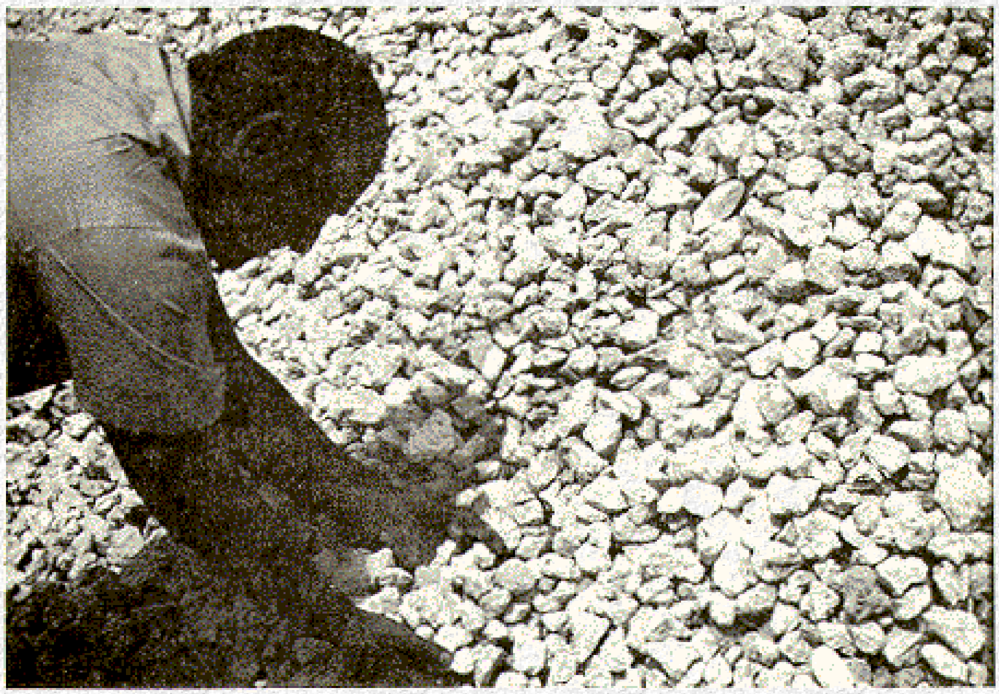

Figure 2-8. Typical Course Rock for Infiltration Trench Backfill *(Courtesy of Dade County Dept. of Public Works, Miami, Florida)*

In addition to utilizing fully perforated CMP, the Florida Department of Transportation has utilized slotted concrete pipe on several South Florida installations. Pipe meeting the general requirements of ASTM C-76 is modified to provide 3/8 inch (9.5 mm) wide slots. The slots are either saw cut after casting or formed in the fresh concrete during casting. The slots are either centered about the springline and staggered on both sides of the pipe barrel by saw cuts (Alternate A) or cast above and below the springline (Alternate B). No significant reduction in strength has been observed using the standardized details shown in Figure 2-9. The design provides sufficient pipe exfiltration rates. Additional slots could be provided when soils with extremely high infiltration rates are encountered. Pipe diameters between 18 inches (0.458 m) and 48 inches (1.22 m) have been used, depending on flow and storage requirements. Although the installations have not been test verified, one 48 inch (1.22 m^2) diameter slotted concrete pipe in a coarse rock trench in a high permeable clean sand apparently exfiltrated runoff from a severe storm without significant discharge from the positive relief drain. The storm deposited approximately 11 inches (0.28 m) of rainfall within a 10 hour period. Controlled field tests using pipe with precast slots have recently verified the performance of this alternate slot detail.

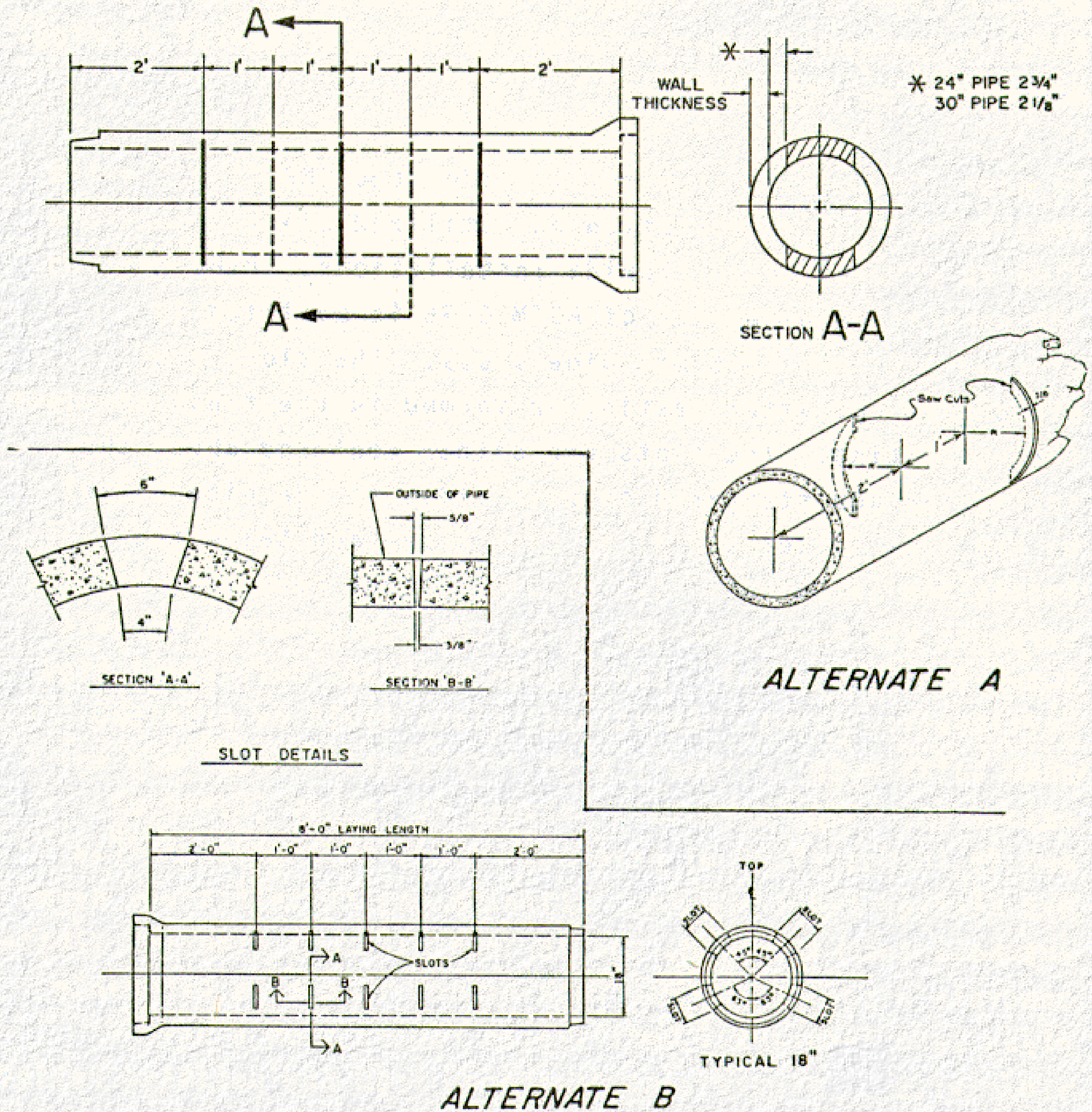

Figure 2-9. Details of Slotted Concrete Pipe *(Courtesy of Florida DOT)*

Determining the size of pipe and trench needed requires an estimate of the surface runoff and a storage volume sufficient to retain this amount of water until it can seep into the adjacent soil, or be released to a conventional storm sewer. The final quantity would be reduced by any detention-exfiltration into the soil that might occur during that interval. Infiltration systems can also be incorporated as part of a positive outfall or combination system to exfiltrate storm water as needed to recharge ground water at various locations along the alignment. Flow can be confined to the conventional storm drain system in areas of the alignment where recharge is restricted by local ordinance. The design of these and other types of subsurface storm water disposal/detention systems are discussed in detail in Chapter 4-C.

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General Considerations

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Preliminary Information

The disposal of storm water by infiltration can provide a practical and attractive alternative to the more conventional and often costlier storm water conveyance systems. Recent legislative mandates lend impetus to consideration of this alternative. The imposition of requirements for zero discharge (zero increase of runoff) within urban areas coupled with regulations on land developments provides an increased emphasis on the infiltration alternative for disposition of storm water.

Infiltration systems provide the designer an additional degree of flexibility in the development of new facilities that avoid additional flow to existing storm drains, or streams and rivers. These systems afford a dual potential in that they are often less costly than conventional systems and they serve to replenish depleted groundwater supplies and increase groundwater levels, preventing undesirable intrusion into aquifers. However, the legal and environmental regulations and soil conditions should be investigated for a particular locality before designing a given system. Governmental agencies should be consulted concerning the amount of aquifer clearance required.

Important sources for information to be considered when determining the feasibility of a particular system are:

- environmental and legal constraints (Chapter 3-A)
- groundwater data (Chapter 3-A)
- local Soil Conservation Service (SCS) maps (Chapter 3-B)
- aerial photos (Chapter 3-B)
- soil boring logs (Chapter 3-B)
- soil properties data (Chapter 3-B)
- rainfall data (Chapter 4-B)

A good source of information is the U.S. Geological Survey-operated National Water Data Exchange (NAWDEX), a cooperative clearing house for water data, including groundwater quality information. NAWDEX assists users of water data to identify, locate, and acquire needed data. Refer to "Status of the National Water Data Exchange (NAWDEX) - September 1977" by M. D. Edwards, U.S. Geological Survey Open-File Report 78-154, 1978.

The following sub-chapters of this manual (3-A, B and C) provide guidelines for selection and evaluation of alternate storm water disposal systems.

A. Environmental and Legal Considerations

1. Introduction

This section discusses the various environmental and legal constraints that should be given consideration in planning and designing underground disposal systems for storm water runoff.

Studies sponsored by the U.S. Environmental Protection Agency, Federal Highway Administration, U.S. Geological Survey, and others, have identified constituents of paved roadways and parking facilities in runoff waters. Assessment of the impact of runoff-conveyed pollutants on receiving waters is continuing (1,2,3,4). Few studies are concentrated on the impact of pollutants in roadway runoff on the groundwater system. Perspective on the possible environmental aspects of subsurface disposal of storm water runoff can be gained from information available on the land treatment of municipal wastewater. Design guidelines for the use of these systems are defined in detail in the "Process Design Manual for Land Treatment of Municipal Wastewater", published jointly by the U.S. Environmental Protection Agency, U.S. Army Corps of Engineers, and U.S. Department of Agriculture (5). In the cover letter to that manual, Jorling and Graves make the following very meaningful statement.

"Wastewater treatment is a problem that has plagued man ever since he discovered that discharging his wastes into surface waters can lead to many additional environmental problems. Today, a wide variety of treatment technologies are available for use in our efforts to restore and maintain the chemical, physical, and biological integrity of the nation's waters.

"Land treatment systems involve the use of plants and the soil to remove previously unwanted contaminants from wastewaters. Land treatment is capable of achieving removal levels comparable to the best available advanced wastewater treatment technologies while achieving additional benefits. The recovery and beneficial reuse of wastewater and its nutrient resources through crop production, as well as wastewater treatment and reclamation, allow land treatment systems to accomplish far more than conventional treatment and discharge alternatives.

"Land treatment processes should be preferentially considered as an alternative wastewater management technology. While it is recognized that acceptance is not universal, the utilization of land treatment systems has the potential for saving billions of dollars. This will benefit not only the nationwide water pollution control program, but will also provide an additional mechanism for the recovery and recycling of wastewater as a resource."

Land treatment of wastewater can provide an alternative to discharge of conventionally treated wastewater. However, careful consideration of any adverse impact of percolated wastewater on the quality of the groundwater is an essential prerequisite for all such projects. It has been demonstrated in numerous reported case histories (5) that a system of disposal which includes filtration through soil can be successful.

The response to a questionnaire circulated for the purpose of eliciting state-of-the-art information for this manual suggests reasons why these systems have not had widespread use. It was indicated that many agencies refrain from using infiltration or subsurface methods for the disposal of storm water to avoid possible adverse impact on groundwater. On the other hand, emphasis is being given in many areas to reduction or elimination of discharge of storm water into surface waters to avert possible pollution, particularly the initial half-inch (13 mm) of runoff, which comprises the "first flush" and carries the highest concentration of surface pollutants(6). This quantity of runoff, however, may vary depending upon development of new information and should not be specified arbitrarily since runoff in excess of one-half inch (13 mm)-may be required to "flush off" surface pollutants. Subsurface disposal provides an alternative method of handling these storm water contaminants.

Like land treatment of wastewater, subsurface disposal of storm water is, an attractive, cost-effective alternative to conventional discharge into surface waters. Consideration of the impact of subsurface disposal of infiltrated storm water on the quality of the groundwater is essential. The quality of groundwater should be determined and compared to established standards for its current or intended use and monitored for change in quality with time.

Proposed U.S. Environmental Protection Agency Proposed (EPA) requirements in the Federal Register, dated April 20, 1979(7), establish the technical criteria and standards to be used in implementing underground injection control programs within individual states. The proposed requirements prevent the use of systems that endanger underground drinking water sources. These regulations establish programs which prohibit any underground injection by either gravity or pressure injection not authorized by State permit. However, some general State rules are allowed without case-by-case permits. "Well injection", as defined under these proposed requirements, is "subsurface emplacement of fluids through a bored, drilled, or driven well; or through a dug well where the depth is greater than the largest surface dimension and a principal function of the well is the subsurface emplacement of fluids".

These systems are classified under the proposed requirements as Type V wells which includes storm water disposed wells, salt-water intrusion barrier wells, and subsidence control wells. Underground sources of drinking water as defined by EPA include, "All aquifers or their portions which are currently providing drinking water and, as a general rule, all aquifers or their portions with fewer than 10,000 parts per million of total dissolved solids [ppm or mg/1 of TDS]".

Before any system is developed for infiltrating water or making any other change in natural runoff, designers should also make sure that the system will not create legal liabilities for the owners. Legal problems cannot always be averted, but developers should be aware of the water laws and codes of practice of their locality.

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2. Environmental Considerations of Runoff Waters

The principal motivation for elimination of storm sewer discharge into surface waters stems from concern over the impact on public health and the aquatic ecosystem. As combined sanitary-storm sewer systems have been identified and direct discharges reduced, attention has focused on the quality of storm water.

Under Section 208 of Public Law 92-500 (Water Pollution Control Act Amendments of 1972) states are developing areawide water quality management plans to identify and mitigate both point and non-point sources of water pollution. Non-point sources of pollution include land development activities, construction, mining, logging, agricultural and silvicultural activities. The nature of the land surfaces over which storm waters flow, i.e., the use to which they are subjected, is widely recognized as one of the key factors of the quality of storm water(8).

Various approaches to the evaluation of storm water quality and its potential impacts are being considered in the development of the 208 plans. Valuable information should be gained by this effort and consideration of subsurface disposal of storm water will undoubtedly be addressed in the various study plans.

As the permit process for discharge of storm water to surface water becomes more stringent in response to Section 208 evaluations, the subsurface disposal of storm water will attract attention as a possible disposal alternative.

The general references for groundwater quality are drinking water standards since many near-surface or water Table Aquifers constitute the main source of public water supplies. For areas affected by salt-water intrusion or locations with naturally poor-quality groundwater, disposal of poor quality surficial storm water is not a serious concern. The EPA proposed drinking water standards are listed in Table 3-A-1.

Table 3-A-1. EPA-Proposed Regulations on Interim Primary Drinking Water Standards, 1975[(9)]

Constituent Characteristic	Value	Reason for Standard
Physical		
Turbidity, units	1[a]	Aesthetic
Chemical, mg/L		
Arsenic	0.05	Health
Barium	1.0	Health
Cadmium	0.01	Health
Chromium	0.05	Health
Fluoride	1.4-2.4[b]	Health
Lead	0.05	Health
Mercury	0.002	Health
Nitrates as N	10	Health
Selenium	0.01	Health
Silver	0.05	Cosmetic
Bacteriological		
Total coliform, per 100 ml	1	Disease
Pesticides, mg/L		
Endrin	0.0002	Health
Lindane	0.004	Health
Methoxychlor	0.1	Health
Toxaphene	0.005	Health
2,4-D	0.1	Health
2,4,5-TP	0.01	Health

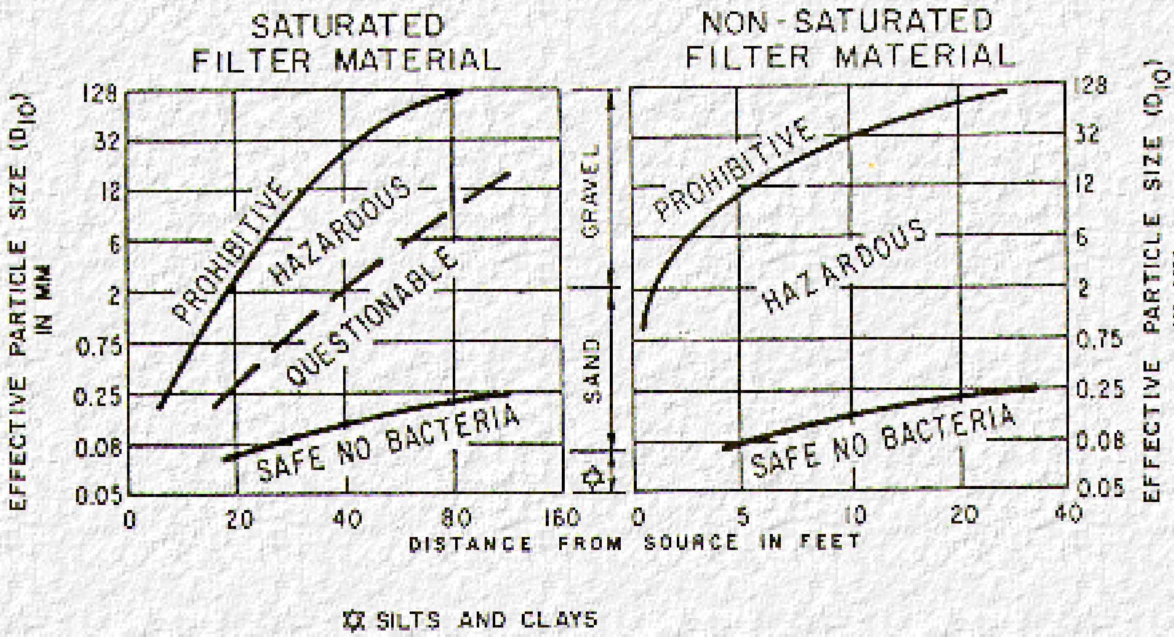
The latest revision to the constituents and concentration should be used.
[a] Five mg/L of suspended solids may be substituted if it can be demonstrated that it does not interfere with disinfection.
[b] Dependent on temperature; higher limits for lower temperatures.

If groundwater contaminants are substantially higher in the area of concern than any of the current listed standards for drinking water quality, future use as a public water supply is doubtful and the subsurface disposal permit process should be greatly simplified.

Most State Health Departments prohibit direct discharge of storm water runoff into underground aquifers. Recharge systems are not utilized in some states because these requirements place restrictions on storm water infiltration systems. Water pollution law in Ohio, for example, can charge offenders with polluting groundwater but those charges must be made and proven in a court of law(10).

Some northern states use large quantities of road de-icing salts during winter months. These states have tended to refrain from use of storm water recharge systems fearing possible contamination of groundwater. To prevent groundwater pollution, some agencies in California require a 10-foot (3.1 m) aquifer clearance for infiltration well construction(11). Infiltration wells are readily capable of polluting groundwater supplies and local regulatory agencies should be consulted concerning the amount of aquifer clearance required for a specific project.

Guidelines are not currently available for aquifer separation distance for infiltration of storm water, However, there are guidelines for sewage effluent from septic tank leach fields. The graphs in Figure 3-A-1 suggest the purification mechanism of soil in terms of distance that effluent must move through various soils for complete removal of bacteria. These graphs indicate that bacteria removal is a function of particle size and groundwater location with reference to filter media. These graphs Cave been used for several years by the State of California for assessing the soil media below dry wells, septic tanks, and leach fields and are based on research conducted by Colorado State University(12,13,14). The graphs are provided in this manual as a guide for establishing separation distances between the bottom elevation of infiltration systems and groundwater level. However, the condition of the storm water entering an infiltration system will probably require less filter media thickness in most cases. Questionable installations should be monitored to identify changes in groundwater quality as discussed herein.



NOTE: THE EFFECTIVE GRAIN SIZE (D_{10}) CORRESPONDS TO THE GRAIN SIZE DIAMETER WHERE 10% OF THE PARTICLES ARE FINER AND 90% COARSER THAN THE EFFECTIVE SIZE BY WEIGHT.

Figure 3-A-1. Sizes of Filter Material Particles that are Effective or Ineffective in Treating Septic Tank Effluent in a Leach Line System [Modified from (14)]

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a. Groundwater Quality Processes

Chemical analyses of water commonly report constituent concentrations as "total". This designation implies that nitrogen for example, is a total of dissolved and particulate phases. The principle dissolved nitrogen species are ammonia, soluble organic nitrogen, nitrite, and nitrate. The particulate phase can be either adsorbed nitrogen, organic matter containing nitrogen, or insoluble mineralogic phases with nitrogen in the lattice.

The particulate phases of the various elements are also represented in the suspended sediments. The distinction is sometimes important as soils and interstitial areas of some aquifers can filter out particulate or suspended solids thereby reducing the impact of the various pollutants on the groundwater. This is particularly important in the case of bacteria.

The natural filtration of runoff water by the soil removes most harmful substances before they can reach the waterbearing aquifer. Nearly all pathogenic bacteria and many chemicals are filtered within 3 to 10 feet (0.9 to 3.1 m) during vertical percolation, and within 50 to 200 feet (15.3 to 61 m) of lateral water movement in some soil formations(15).

Tests made by the U.S. Department of Agriculture for the Fresno Metropolitan

Flood Control District, indicated heavy metals such as lead, zinc, and copper present in the upper few centimeters of storm water infiltration basin floors. Generally after 10 to 15 years of storm water collection, this layer may require removal or other treatment where a build-up of concentrations of these elements has occurred. The particular locations tested by the U.S. Department of Agriculture had soils with a relatively high clay content(10). Layers of fine sands, silts, and other moderately permeable soils also very definitely improve the quality of storm water. This concept underlies the practice of disposing of domestic sewage in septic tanks with leach lines or pits and the land disposal techniques.

One of the major traffic-related contaminants is lead. Although lead is primarily emitted as particulate matter, it is fairly soluble. Lead in its ionic form, tends to precipitate in the soil as lead sulfate and remains relatively immobile due to low solubility(l6). Lead can also be tied up by soil microorganisms, precipitate with other anions, ion exchange with clay minerals, or be absorbed by organic matter or uptake by plants. Once ionic lead reaches the groundwater table by precipitation, ion exchange, or adsorption the available lead can still be reduced. Surface and groundwater quality samples collected near a major highway interchange in Miami, Florida, revealed that lead concentrations were very low(17). The interaction of lead with the high bicarbonate in this particular location probably caused precipitation in the surface water borrow pond near the highway. Lead concentrations in the bottom sediments of these ponds were found to be relatively high.

If impure water is allowed to enter directly into coarse gravel or open joints in rocks, the impurities may enter into and contaminate adjacent groundwaters. Sites that are underlain with highly permeable strata or cracked and jointed rocks have the best capabilities for rapid disposal of surface waters. Unless adequate arrangements are made to treat contaminated water, or to filter impurities, infiltration systems may degrade the grounds water quality. Faults and intrusions, should always be evaluated for their effect on groundwater occurrence, influence on quality, and direction of movement. If the underlying rock strata is fractured or crevassed like limestone, storm water may be diverted directly to the groundwater, thereby receiving less treatment than percolation through soil layers.

Breeding and Dawson(18) describe a system of 127 recharge wells used by the City of Roanoke, Virginia, to dispose of storm runoff from newly developing industrial and residential areas. Several major faults exist in the underlying bedrock. These faults play a significant role in the effectiveness of the drainage wells, and also in the movement of groundwater. The authors also indicate that these direct conduits to groundwater have caused quality degradation in one area; however, "groundwater users in adjacent Roanoke County have not experienced quality problems that could be connected to this means of storm water disposal."

The case cited illustrates the possibility of groundwater contamination in areas where fractured and highly permeable rock layers exist, providing conduits for widespread movement of contaminants. It is, therefore, important in the planning stages of a large subsurface storm water disposal project to identify the underlying soil strata in terms of its hydraulic, physical, and chemical characteristics. Pertinent physical characteristics include: texture, structure, and soil depth. Important hydraulic characteristics are: infiltration rate and permeability. Chemical characteristics that may be important include pH, cation-exchange capacity, organic content, and the absorption and filtration capabilities for various inorganic ions. If detailed groundwater quality analyses are available, it is possible to compute the solution-mineral equilibrium(19). This approach does not guarantee that an anticipated chemical reaction will occur but does indicate how many ionic species should behave.

The items referring to physical and hydraulic characteristics are addressed to some extent in other chapters of this manual. Further discussion of the chemical characteristics of soils is beyond the scope of this manual. Definitive information on this subject can be obtained by consulting appropriate references, i.e., Grim(20), or other references on the subject. The importance of proper identification of the hydraulic characteristics of the rock strata has been illustrated above.

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b. Groundwater Monitoring

Environmental laws and regulations now in force require monitoring groundwater where adverse effects to its quality may result from disposal and storage of solid and liquid wastes(21). Monitoring systems have not as yet been required for groundwater recharge utilizing storm water. However, consideration of such monitoring systems should be incorporated in the design of subsurface drainage systems that discharge storm water directly into groundwater.

Proposed EPA requirements for Type V wells (gravity or injection), which discharge directly into surficial aquifers, call for immediate action with respect to injection that poses a significant risk to human health. An assessment is required of the contamination potential, available corrective alternatives, and their environmental and economical consequences(7).

When properly installed, a groundwater monitoring system should provide sufficient data for determining the extent of contamination buildup with time, as well as concentration and distribution of the contaminants.

Geologic analysis of the area can provide vital information for developing the monitoring system. Factors to be considered include: depth and type of subsurface soils, depth to bedrock, relative permeabilities, depth to groundwater, and relative groundwater gradients. Proper layout of monitoring wells cannot be accomplished until information relative to such factors has been obtained and evaluated. Wells must be sufficiently close to the potential source of contamination to detect any degradation of groundwater quality at an early stage. Where monitoring wells are used as an early warning system, it is imperative that the preproject quality of on-site groundwaters be established, and, thereafter employed as a standard for comparison with groundwater samples taken subsequent to initiation of the proposed subsurface drainage system. Sufficient samples of groundwater should be obtained over a time period adequate to establish the "ambient" groundwater conditions prior to storm water disposal. The number and location of monitoring wells will be governed by the magnitude of the project and careful consideration of information developed by the aforementioned site geology analysis.

An appropriate monitoring well should be so designed as to provide the quantity and quality of sample required at the lowest cost. Small diameter (1 1/2 inch [38 mm]) PVC riser pipe, with either plastic well screens or slotted plastic pipe, will usually prove adequate in developing a sampling well. Slotted pipe is the least expensive and most convenient material for developing a suitable well screen(21). Materials used in the construction of the sampling well should be chosen so that they do not influence the characteristics of the sample.

To prevent the migration of fines into the sampling well, all well screens or slotted sections should be installed with a backfill of clean filter sand. Precautions should be taken to prevent the migration of fines into the wells. The top portion of the well pipe should be backfilled with concrete or cement grout to provide a seal which prevents contamination by surface waters. The well seal should comply with State and local requirements.

A shallow well groundwater quality monitoring system has been developed in Southern Florida which will be installed routinely as a contract item on infiltration trench projects in Dade County. Details of this system are similar to the cross-section and plan shown on Figures 3-A-2 and 3-A-3.

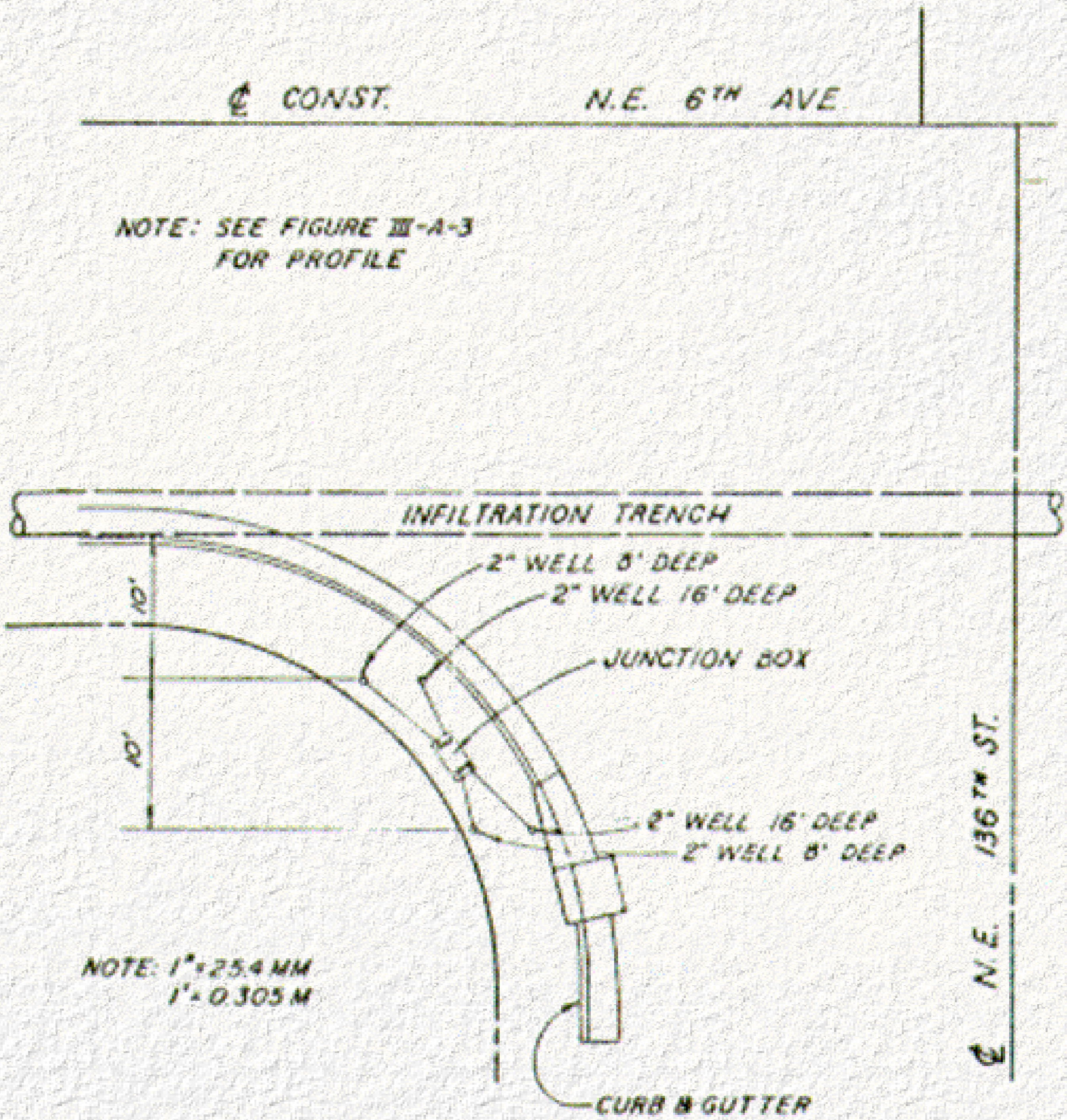

Figure 3-A-2. Plan View of Groundwater Monitoring System for Infiltration Trench Construction ***(Courtesy of Bristol, Childs & Associates, Coral Gables, Florida)***

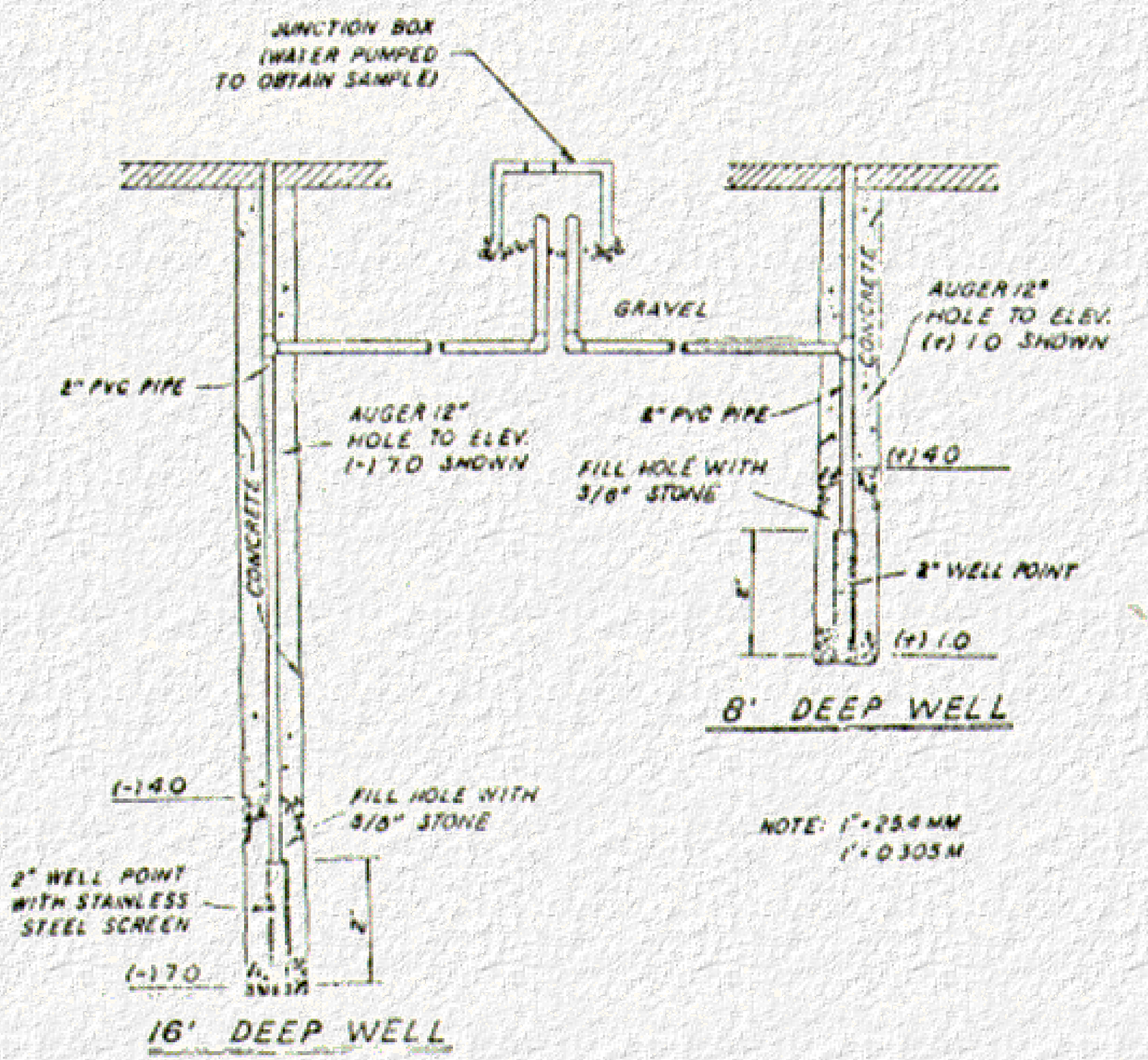

Figure 3-A-3. Typical Groundwater Monitoring Wells for Infiltration Trench Construction
(Courtesy of Bristol, Childs & Associates, Coral Gables, Florida)

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3. Legal Considerations

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a. Introduction

Before any system is developed for infiltering water or making any other change in natural runoff, designers should make sure that the system will not create legal liabilities for the owners. Major construction projects can change the natural runoff patterns, reducing flows in some areas and increasing it in others. Areas that had no known record of flooding before the construction of a major work may subsequently develop drainage problems. Often the increased discharges can be attributed to "improvement" of the natural delivery system, rather than diversions. In other areas, farmers or others who have depended on natural flows in streams for their livestock or crop production, see the available supplies sharply reduced. In semi-arid areas, the construction of detention ponds, seepage pits or wells, catch basins, reservoirs, etc., for "water harvesting" has reduced the flows to downstream landowners. Such changes can lead to litigation. Legal problems cannot all be averted. Developers of systems should contact appropriate local or state agencies regarding compliance with laws or local codes of practice.

Drainage of surplus storm water from changing land use and development may cause increased erosion with resultant pollution in natural waterways. Relatively new political constraints have been imposed because of this and burgeoning public sensitivity to further environmental degradation. Levels for various constituent concentrations in discharge or receiving waters may be specified in permits to maintain water quality objectives. Legislation specifying zero discharge and zero increase in discharge has been enacted in some cases without provision for exceptions, despite their merits, environmental or otherwise.

Zero increase in discharge may be a difficult legal concept. It attempts to recognize the need for runoff and provide for engineering flexibility. However, legal problems will arise from interpretation of runoff coefficients. A coefficient by definition is a ratio or, as commonly expressed, a percentage figure. Even in a natural watershed, with excellent rainfall and runoff (discharge) records, the runoff coefficient has been shown to vary with the rainfall frequency, rainfall intensity or rate, period of antecedent dry conditions (soil moisture content), and the seasonally dependent vegetation. When comparing areas, the infiltration rate (percolation) of the soils, the size of the area, the degree of imperviousness (roads and roofs, etc.), the slope, and vegetation type, become important. It would be difficult to anticipate a runoff coefficient with a high degree of confidence for an area that is to be altered with respect to these variables.

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b. Water Rights

When subsurface drainage systems are to be employed, consideration must be given to their effect on water rights downstream, or senior claims to the water, as the source of flow will be diminished when the runoff is diverted from its normal or historic drainage channel (22). If the concept of "zero" increase in runoff is pursued, no interference in downstream rights would be anticipated.

The Process Design Manual for Land Treatment of Municipal Wastewater(5) points out that water rights problems tend to arise in either water-deficient areas or those areas fully allocated.

Most riparian (land ownership) rights are in effect east of the Mississippi River, while most appropriation (permit system) rights are in effect west of the Mississippi River.

Legal distinctions are made between discharges to a receiving water in a well-defined channel or basin (natural watercourse), superficial waters not in a channel or basin (surface waters), and underground waters not in a well-defined channel or basin (percolating or groundwaters)(23).

Transportation-related aspects of water rights are discussed in "AASHTO Guidelines for the Legal Aspect of Highway Drainage"(24).

Possible water rights problems related to complex drainage systems may require consultation with water masters or water rights engineers at the State or local level. An excellent reference is the National Water Commission publication, "A Summary-Digest of State Water Laws"(25). Similar case histories can be found in references(22,23, 24,25). The assistance of an attorney versed in water law is often helpful.

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4. Summary and Conclusions

. Since the character and concentration of pollutants generated from paved surfaces vary considerably depending upon the type of development, location, population, and dilution by storm water runoff, no attempt is made in this manual to define these constituents and evaluate their effects on the environment. Various studies are underway at the present time which address this problem.

b. Land treatment of storm water by infiltration through soil is capable of removing pollutants at levels comparable to the best available advanced wastewater treatment technologies. This capability will vary with the hydraulic, physical, and chemical characteristics of the receiving soil strata and the character and concentration of the pollutants carried by the storm water.

c. A monitoring program may be required to determine the quality of groundwater and compare it to established standards for current or intended use, and to evaluate any potential for degradation with time. It is, therefore, advisable to consult state and local regulatory agencies in regard to environmental and legal questions relative to subsurface disposal systems for storm water.

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B. SOILS EXPLORATION

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1. Considerations for Determining Subsurface Soil and Groundwater Conditions

A key element in any design analysis of soil infiltration capacity undertaken for a subsurface storm water disposal system is a comprehensive soils investigation program, supervised by a Soils Engineer qualified to plan and implement the program and interpret the results. Valuable professional assistance or guidance may be available from governmental agencies such as the Soil Conservation Service, U.S. Department of Agriculture. A hydrogeologist knowledgeable with the local geohydrology could also provide valuable information. The details of subsurface exploration programs related to this subject are beyond the scope of this manual. However, any soils exploration program should be oriented to the following objectives:

. Define the subsurface profile within the infiltration basin or well area; or along the length of the proposed system.

  1.) Identify soil and rock strata, 2.) locate the otatic water table, and 3.) anticipate its seasonal fluctuations

b. Provide representative samples from the explorations for laboratory testing purposes.

c. Provide for field permeability tests to be performed at the site as necessary. For suggested methods refer to Chapter 4-A of this manual.

d. Review data on historic and existing groundwater conditions to provide information on possible mounding effects of the proposed system.

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2. Preliminary Activities

This phase of an investigation can be categorized as a reconnaissance study since a great deal of subsurface information is frequently available from various sources. Available data can often be acquired at little or no cost. Its acquisition can provide insight into existing conditions and aid in determining the extent of the subsurface explorations program needed for final design.

---

a. Possible Sources of Existing Subsurface Data

1. Soil surveys prepared by the Soil Conservation Service, U.S. Department of Agriculture, are available for all states, as well as Puerto Rico and the Virgin Islands. The soil surveys published subsequent to 1957 contain interpretations of the mapped soil deposits useful for engineering purposes, including soil suitability for drainage and irrigation. Soil surveys prior to 1958 require more engineering interpretation. Copies of these surveys can also be inspected in Soil Conservation District or County Agricultural Extension Offices.

2. Geologic reports and groundwater resource reports prepared by the U.S.

Geological Survey in cooperation with state agencies are frequently available. These reference sources can be quite informative.

3. Subsurface data obtained previously in the area for other projects should not be overlooked. Such data may have been obtained in connection with utility company projects; or private ventures such as commercial developments, or earlier public agency projects.

4. When available, aerial photographs can also be of value when properly interpreted by trained personnel in defining soil type categories and qualitative soil-moisture conditions.

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b. Preliminary Site Inspection

A great deal can be learned about sites in non-developed areas through examination of the terrain and its surface features. Types of vegetation and lake levels may give some preliminary indication of groundwater levels. The natural terrain is indicative of land forms, which in turn imply the types of soil categories that exist. Soil maps and bulletins of the U.S. Department of Agriculture's Soil Conservation Service are most helpful in the interpretation of information derived from such on-site examinations. Commercial or residential construction records can also provide information on soil and groundwater conditions. Inspection of existing wells may yield general groundwater data.

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3. General Guidelines for Explorations Programs

The subsurface explorations and field testing program should be established after a review of the data obtained in the reconnaissance phase. The storm water disposal system might be only part of a larger project that has its own explorations requirements. Explorations should be made to serve dual purposes whenever possible. Those required specifically for storm water disposal can be planned after due consideration and evaluation of existing data and, where applicable, considering explorations requirements for other project design features.

A preliminary program of limited scope can help establish groundwater and soil types and aid in verifying the information obtained in the reconnaissance stage, or serve, itself, as the reconnaissance stage where other data is not available. A preliminary program is advisable whenever possible since it might well indicate at an early stage whether a subsurface storm water disposal system is feasible or not. In addition, the magnitude of explorations for final design would be more apparent following a preliminary program.

Explorations can be implemented in various ways, such as machine-cased borings, test pits, trenches, or auger holes. Penetration resistance is not considered an applicable exploration method.

Although an in-depth discussion of types of exploration and methods is not within the scope of this manual, some comments on selecting exploration types and procedures are appropriate. Test pits or trenches are often the best methods since they expose soil and water conditions. Pits or trenches may also be the most economical method depending on the equipment and manpower available to the designer. Where cased borings are used, they should be made to the maximum depth possible without the use of water to facilitate determination of natural groundwater depths. In addition, cased borings, or auger holes, should provide continuous samples to some depth below the final bottom elevation of the proposed infiltration trench, well, or basin. This is necessary in order to establish a continuous definition of soil types through which the storm water will percolate and to aid in determining groundwater depth through differences in soil moisture. The recommended minimum depth of exploration is 10 feet (3.1 m) below the bottom of the seepage discharge level, or to the static water table, whichever occurs first.

An example of a typical subsurface exploration program for basin design and for trench design are shown on Figure 3-B-1 and Figure 3-B-2, respectively. The finished grade for the basin example in Figure 3-B-1 and for the trench example in Figure 3-B-2 are less than 4 ft (1.22 m) above the static water table in sand and gravel materials. An adjustment of these grades may be desirable to satisfy local environmental considerations. Mounding conditions above the water table should be anticipated in both cases with some reduction in infiltration capacity.

Long term readings are essential to evaluate seasonal groundwater fluctuations, particularly where the groundwater level may be within 10 feet (3.1 m) of the seepage discharge level. This information can be obtained by inserting perforated or slotted tubing or pipe in the boring and taking periodic water level readings.

An area with a high groundwater table and/or soils having a high percentage of silt and clay size material will not normally accommodate subsurface storm water disposal. In such areas a storage-retention type of system should be considered as an alternative. There are exceptions to this, since, in some areas, a high water table with pervious soil conditions may not be detrimental to the use of subsurface disposal systems.

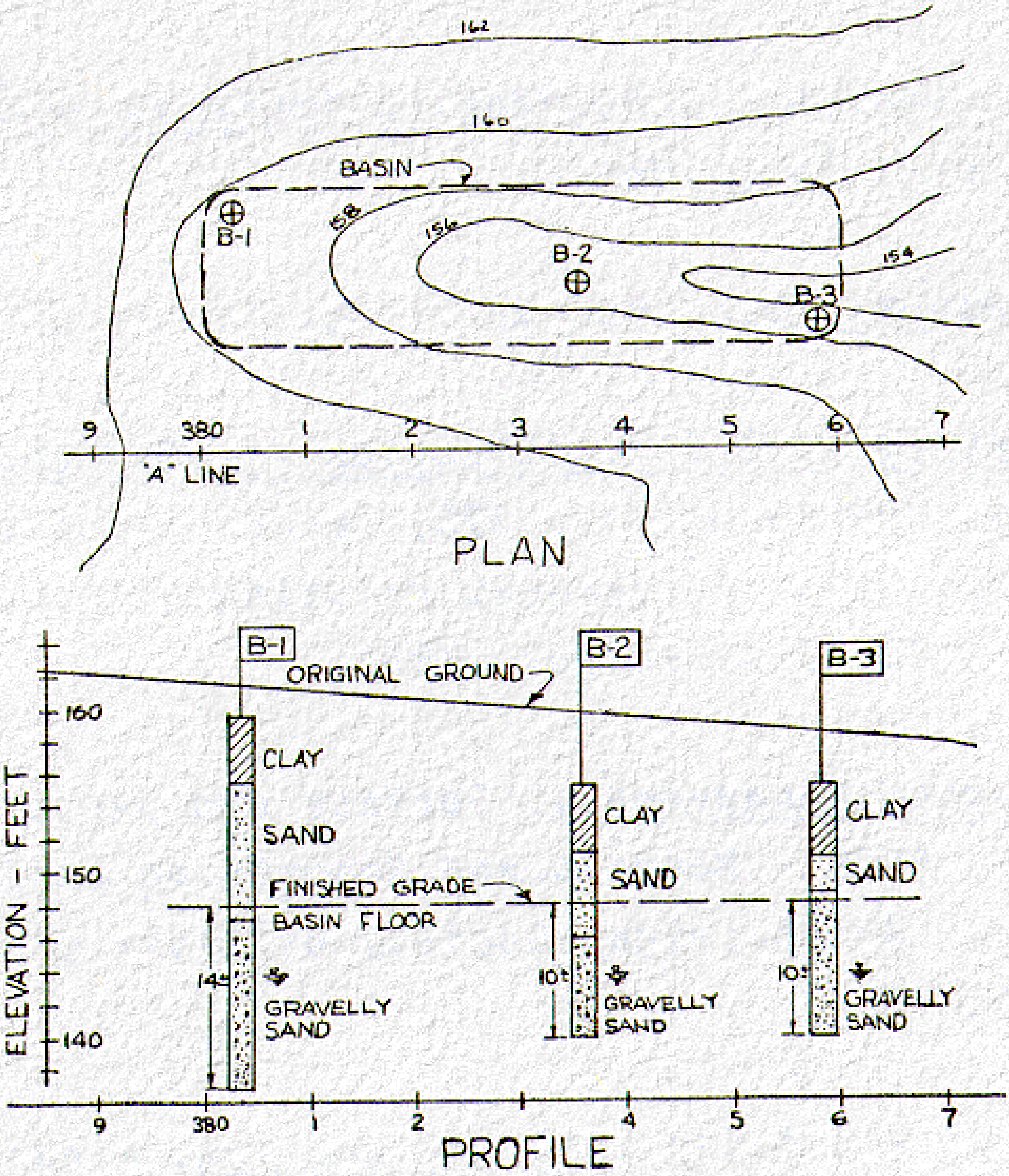

Figure 3-B-1. Typical Exploration Program for Infiltration Basin Design

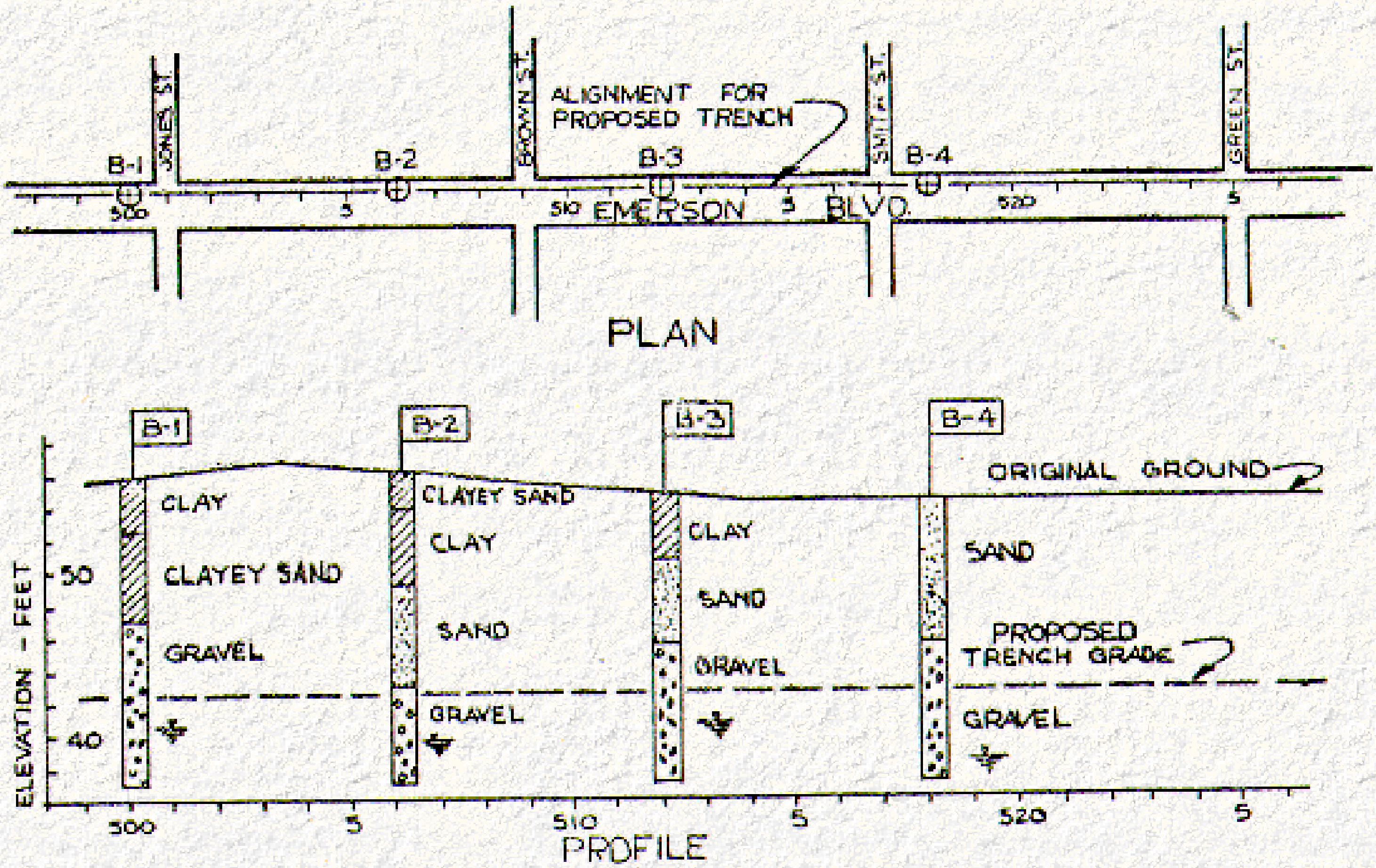

Figure 3-B-2. Typical Exploration Program for Infiltration Trench Design

References

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C. Evaluation of Alternative Disposal Systems

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1. Alternatives to Positive Discharge

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a. Basins

When ample space is available and other criteria are satisfied, the use of infiltration-basins can provide a relatively inexpensive solution to storm water disposal in terms of cost per unit volume of water drained. As mentioned earlier, space is often available within areas of the right-of-way, such as highway interchanges; or on non-used portions of residential or commercial developments.

In some communities infiltration basins have been integrated with attractive parks and/or recreation areas. This dual role of the basin benefits both the facility and the public.

Among the negative aspects of infiltration basins are their susceptibility to early clogging and sedimentation, and the considerable surface land areas required for their construction. Basins also present a security problem due to exposed standing water, and a potential for insect breeding. These problems are discussed at length in Chapter 6 of this manual.

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b. Wells and Pits

Wells and pits are often used to handle drainage problems in small areas where an outfall is not available. They are also used in conjunction with infiltration basins to penetrate impermeable strata overlaying pervious soil layers. Infiltration wells and pits can be installed quickly and inexpensively to remove standing water in areas difficult to drain. A disadvantage is the tendency of filter media to clog with silt or sediment, requiring considerable maintenance. Also, their capacity for drainage is difficult to predict. One well may induce a good rate of infiltration; while another, a very short distance away, will drain very poorly.

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c. Trenches

Infiltration trenches are a viable solution for long-term underground storm water disposal at locations having soils or rocks capable of absorbing large quantities of water. Trenches are ideally suited to urban development; e.g., under lot lines, within easements, under road right-of-way, beneath parking lots and in landscaped areas.

Slab-covered trenches and trenches with perforated or slotted pipe backfilled with coarse aggregate provide economical alternatives to surface disposal. The slab-covered trench is feasible where rock strata will support the slab and trench walls and still provide necessary infiltration. Such conditions are found in certain areas of Florida although this particular design may have limited application elsewhere.

Perforated or slotted pipe backfilled with coarse rock, installed in trenches, can provide a long-term solution to underground storm water disposal. The capacity of this system is controlled by the native soil permeability characteristics. The pipe provides storage and also serves as a continuous catchment for silt. Clogging of perforations or slots and coarse aggregate is thus minimized. Catch basins which are points of entry of storm water also provide silt catchment and easy access for cleanout.

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d. Combination Systems

These systems can incorporate retention storage with subsequent infiltration and discharge of residual flow through a positive outfall system. Individual systems can be designed to infiltrate storm water along the entire alignment of the drainage system or to infiltrate water only in selected areas.

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e. Economic Considerations

Excavation materials from infiltration basin areas or trenches can provide a savings by their utilization in the construction of embankments. Considerable savings are also possible by reducing or eliminating costly outfall facilities. Local drainage problems can also be solved in some areas by installing sumps and drilling dry wells to take advantage of the infiltration characteristics of the soil and reduce storm drain requirements. In order to evaluate the economic feasibility of a given design, in addition to initial cost, the long term maintenance requirements are an essential consideration.

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2. Site Evaluation and Selection of Alternative Infiltration Systems

The following is a check list of the steps which should be included in a feasibility evaluation:

. Potential Benefits

1. Economic benefits compared to direct discharge (positive system).

      . Reduced outflow requirements.

      2. Reduced need for treatment of storm water.
   2. Groundwater Recharge

   3. Reduced or zero increase in discharge

   4. Reduced subsidence due to Groundwater withdrawal

   5. Reduction or prevention of salt-water intrusion
b. Evaluate alternate systems based on constraints.
   1. Environmental
      . Local impacts

      2. Groundwater quality
   2. Legal

   3. Physical site
c. Evaluate site characteristics
   1. Soil (surface and subsurface)
      . Type and depth of soil

      b. Infiltration characteristics

      c. Location of Groundwater table
   2. Hydrologic
d. Select most feasible system based on:
   1. Economic evaluation

   2. Construction evaluation

   3. Maintenance evaluation

   4. Potential benefits (2.a. above)

   5. Constraints (2.b. above)

   6. Site characteristics (2.c. above)
e. Design system

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A. Determination of Infiltration Rate

1. Factors Affecting Infiltration Rate

The capabilities of sites to accept surface water and distribute it into groundwater systems depend on a great many factors. Among the most important are: natural ground slope, type and properties of surface and subsurface soils, geologic conditions, and subsurface hydrologic conditions. The amount of water to be distributed and the kinds and amounts of contaminants and dissolved matter in the water have a profound influence on the capacities of systems to accept and distribute water on a long-term basis. Dissolved salts and other chemical substances, oil, grease, silt, clay, and other suspended matter can clog the surfaces through which water must enter a system. Such materials will greatly reduce infiltration rates if they are not intercepted by catchment basins or frequently removed by appropriate maintenance methods. The depth of the water table, and its natural slope, as well as the unsaturated and saturated horizontal and vertical permeabilities of soil formations, have important influences on rates of inflow and the rate of buildup of saturation mounds under infiltration systems.

For simplicity, the words: permeability, infiltration, and percolation, are used interchangeably in this manual in describing the ability of soil to absorb water. Specific definitions are included in the Glossary Section of this manual.

Investigations for the design of infiltration systems should concentrate on the following vital aspects of infiltration and dissipation of water: (1) the infiltration capabilities of the soil surfaces through which water must enter the soil, (2) the water-conducting capabilities of the subsoils that allow water to reach underlying water table, (3) the capabilities of the subsoils and underlying soils and geologic formations to move water away from the site, and (4) flow from the system under mounding conditions at maximum infiltration rates.

The rate of infiltration is greatly affected by the permeability of the soil formations. The infiltration rate for the first application of water in an infiltration test is generally greater than, after longer application of water. As water application continues and the uppermost sediments become saturated, the infiltration rate gradually decreases and reaches a nearly constant rate, usually within a few hours. If all of the sediments are uniform or the deeper sediments are more permeable than those near the surface, and the water table is at considerable depth, the infiltration rate is controlled by the sediments near the surface. However, when the deeper formations are less permeable than the shallower ones, the shallow sediments soon become saturated and the resultant infiltration is controlled by the less permeable sediments at greater depth and groundwater gradients under these mounding conditions.

The principles of infiltration have been studied by many investigators, some of whom are referenced at the end of this chapter. One of the most complete studies of the waterflow patterns below infiltrometers is that of Aronovici in 1955(1), who illustrated the significance of surface and subsurface conditions on observed infiltration rates. His study suggested also that pressure head is the dominant factor involved in filtration rates in initially dry or damp soils, and emphasized the influence of the differential hydraulic head in causing a decrease in infiltration rate with time.

Compaction of the exposed surface of a test area reduces the infiltration rate. Wisler and Brater in 1949(2) pointed out that rain beats down on an unprotected soil, compacts it, washes fine debris into the pores, and thereby reduces the permeability.

Musgrave and Free in 1937(3) found that even slight water turbidity caused a considerable decrease in infiltration rate. According to the U.S. Salinity Laboratory in 1954(4), water having the same quality as that to be used later in actual infiltration should be used for the infiltration test.

Weaver(5) indicates that "Infiltration into pervious unsaturated or dry materials is predominantly controlled by capillary suction similar to the process of capillary rise except that gravity assists rather than impedes downward flow." He adds: "While it was long assumed that Darcy's law was valid to deal with these problems of unsaturated flow, this was not proved until 1950, by Childs and Collis-George(6). The difference from the usual applications of Darcy's law is that neither the conductivity term nor the driving potential term are constants; both are functions of water content." He notes that infiltration basin efficiency being directly proportional to the operating head, there is a definite advantage in designing for operation at relatively high heads. For an infiltration analysis, values of four soil properties must be obtained with depth in the profile. Weaver lists these properties as: (1) capillary suction, (2) transmission zone water content, (3) saturated permeability, and (4) soil porosity.

In describing New York's method of analysis of infiltration from basins Weaver states that it is necessary to "Plot and summarize all subsurface and laboratory test data in the same general fashion as for all other types of foundation design problems. These data must then be studied in terms of significance with respect to infiltration theory, to deduce the value of the soil properties controlling the infiltration rate at the site. As a general rule, the control zone for the infiltration rate will be in the first 10 feet (3.1 m) of the uppermost soil layer. The soil properties outside this area may exert a secondary control only when the soil is markedly less permeable and the profile is such that lateral spread of the wet front is prevented if its vertical advance is impeded by this layer. In other words, the surface control zone -- where the primary transmission zone is established -- will control infiltration under any condition where the water transmitted through it has someplace to go, either vertically or laterally."

No soil layer that has a hydraulic conductivity less than the soil within the lower limits of the infiltration facility should be overlooked as a possible zone which would result in groundwater mounding. Mounding over these zones could drastically reduce the infiltration rate of a proposed facility under perched or high groundwater conditions.

Since many factors affect infiltration rates, considerable judgment and experience are needed for selection of the proper test procedure to obtain reliable results from which to design an infiltration system. To interpret infiltration data properly the investigator must know the hydrology of the deep as well as the shallow formations. Adequate subsurface explorations as discussed in Chapter 3-B of this manual should always accompany infiltration tests.

Some typical infiltration (permeability) rates for the various soil groups of the unified soil classification system are given in Table 4-A-1, for saturated and compacted laboratory specimens. Since laboratory test specimens are mixtures of disturbed materials, the tests may give permeabilities lower or higher than those of the in-place materials. If the in-place materials are dense, uniform deposits, and the laboratory specimens are less dense, the laboratory permeabilities could be too high. But if the natural deposits are stratified (sorted) formations, the laboratory permeabilities can be too low. The wide ranges in permeability values in Table 3-A-1 (even for relatively similar materials) emphasizes the need for good subsurface explorations and field permeability and infiltration tests.

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2. Methods for Determining Soil Permeability

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a. General Discussion

Those working with infiltration systems often make use of permeability tests, which are intended to measure a soil's ability to infiltrate water. These tests should simulate as closely as possible, the conditions that will develop in an infiltration system, and presume that each square foot of basin, trench, etc., will infiltrate the rate determined by the test. The value of these tests, therefore, depends on the degree to which they simulate the real conditions.

Table 4-A-1. Permeability Rates for Different Soil Groups for Saturated and Compacted Laboratory Specimens (7)

Major Divisions		Group Symbols	Typical Names of Soil Groups	Unit Dry Weight lb per cu ft		Permeability (K)* and Percolation Characteristics when Compacted and Saturated	
				Std. AASHTO	Mod. AASHTO	cm per sec	ft per day
Course-Grained Soils	Gravel and Gravelly Soils	GW	Well-graded gravels Gravel-sand mixtures little or no fines.	125-135	125-140	10^{-1} to 10^{-4} Pervious	300 to 0.3
		GP	Poorly graded gravels or gravel-sand mixtures little or no fines.	110-125	110-140	10 to 10^{-2} Very Pervious	$3x10^{-4}$ to 30
		GM	Silty gravels, gravel-sand-silt mixtures.	115-135	115-145	10^{-3} to 10^{-6} Semi-pervious to impervious	3 to $3x10^{-3}$
		GC	Clayey gravels. gravel-sand-clay mixtures.	115-130	120-145	10^{-6} to 10^{-4} Impervious	$3x10^{-3}$ to $3x10^{-5}$
	Sands and Clays	SW	Well-graded sands, gravelly sands, little or no fines.	105-120	110-130	10^{-2} to 10^{-4} Pervious	30 to 0.3
		SP	Poorly graded sands or gravelly sands, little or no fines	100-120	105-135	10^{-1} to 10^{-3} Pervious	300 to 3
		SM	Silty sand, sand-silt mixtures.	100-125	100-135	10^{-3} to 10^{-6} Semi-pervious to impervious	3 to $3x10^{-3}$
		SC	Clayey sands, sand-clay mixtures.	105-125	110-135	10^{-6} to 10^{-8} Impervious	$3x10^{-3}$ to $3x10^{-5}$
	Silts and Clays LL is less than 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	85-115	90-125	10^{-3} to 10^{-6} Semi-pervious to impervious	3 to $3x10^{-3}$

Fine-Grained Soils		CL	Inorganic clays of low to medium plasticity gravelly clays, sands clays, silty clays, lean clays.	90-120	90-130	10^{-6} to 10^{-8} Impervious	$3x10^{-3}$ to $3x10^{-5}$
		OL	Organic silts and organic silty clays of low plasticity.	80-100	90-105	10^{-4} to 10^{-6} Semi-pervious to impervious	0.3 to $3x10^{-3}$
	Silts and Clays LL is greater than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	70-95	80-105	10^{-5} to 10^{-7} Semi-pervious to impervious	0.03 to $3x10^{-4}$
		CH	Inorganic clays of high plasticity, fat clays.	75-105	85-115	10^{-6} to 10^{-9} Impervious	$3x10^{-3}$ to $3x10^{-6}$
		OH	Organic clays of medium to high plasticity, organic silts.	65-100	75-110	10^{-4} to 10^{-8} Impervious	$3x10^{-3}$ to $3x10^{-5}$

*Permeability values as modified by H.R. Cedergren

Soil permeability or infiltration rate is best determined by actual field tests under known hydraulic gradients and known seepage areas. The value of laboratory tests is limited to the degree to which the specimens tested actually represent the soil mass in the field. One of the more important factors influencing the permeability of a soil of a given grain size distribution is the porosity and structural arrangement of the grain particles. In laboratory test specimens both properties are likely to be disturbed during sampling or during test preparation. Also, soil formations are stratified to a greater or lesser degree, and are variable within a formation; hence it is best to determine permeabilities in the field since a large zone of influence can be tested with less error.

Because of the possibilities of error introduced by laboratory permeability testing, as noted above, it is suggested that such tests be used only as a guide for preliminary evaluation of proposed infiltration drainage sites.

Field methods should be used to simulate conditions that most nearly predict the drainage capability of the proposed drainage system. This can be accomplished by auger holes, (cased or uncased), and sample trenches or pits; or other field procedures. The method chosen will depend on the type of facility to be designed and on the site location parameters; i.e., presence of underground utilities, number of test sites required, requirements for maintenance of vehicular and/or pedestrian traffic, type of equipment available to perform the test excavation, and type of subsoil.

The size of the test excavation should be large enough to aid in visual inspection when possible and to provide sufficient surface area to distribute the water, either laterally or vertically, depending on the type of test performed. Normally 12-inch to 24-inch (0.3 to 0.6 m) width or diameter is sufficient to accomplish this with testing equipment available. Longer test areas or excavations may be required for basin or pit testing.

The number of test sites is somewhat dependent on existing soil conditions and the drainage system layout.

For a basin, or a subsurface system for a paved parking lot area 300 ft x 300 ft (92 m x 92 m), two or three tests would normally be sufficient. On a continuous linear trench system of 1/2 mile (800 m) or more, 500 foot (150 m) intervals between test locations is sufficient, provided soil is uniform in composition.

Tests should be performed at each distinct change in soil strata and should continue downward to the approximate bottom elevation of the drainage facility being designed. If test results indicate low infiltration rates, excavation and testing should be continued to a depth that would provide satisfactory infiltration and yet still be economical for construction of the drainage facilities and within compliance of local and state regulations as defined in Chapter 3-A, "Environmental and Legal Considerations".

An adequate supply of water should be available to both presoak the sides of test excavation or auger hole and perform testing. This can be supplied by either truck, hose, or fire hydrant. Excavation equipment may be either auger, backhoe, or trenching machine. A timing device with a second hand is needed for performing the test. Backfill material should be available to cover the excavation when testing is completed.

Test data should be recorded in a form that can be easily analyzed in the field to determine if the results are satisfactory to accommodate the design drainage facility.

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b. Indirect Methods

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1. SCS Soil Classification Maps

These are maps that give the SCS classification of surface soils in many parts of the United States. They are published in National Cooperative Soil Survey Reports published by the Soil Conservation Service, U.S. Department of Agriculture, in cooperation with other agencies. Soil survey information is available on a county by county basis. A portion of a typical map is shown in Appendix D-1. These maps cannot possibly cover variations occurring in short distances; they give only a general idea of the basic types of soils occurring in various areas. Any use of these maps to catalogue soil type for estimating permeability should be verified by actual field inspection and classification of soils in the study area. Such maps can indicate in a general way whether soils might be expected to have good drainage, moderate drainage, or very poor drainage. Therefore, they may be utilized to some extent in preliminary infiltration drainage feasibility studies. Before any system is designed, more specific information based on field permeability testing should be obtained for a given site.

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2. Specific Surface Method of New York State(8)

This method (Appendix D-2) is used only for cohesionless granular material that is uniform and non-stratified. The saturated coefficient of permeability is calculated with a formula developed empirically which relates porosity, specific surface of solids, and permeability. Its principal advantage is simplicity. It requires only a small number of samples of material to obtain a standard gradation, the shape characteristics of the grains contained in each sieve size interval, and calculation of the specific surface based on the data obtained from the grain size analysis and physical examination and an estimated in-place porosity. As with other indirect methods, it does not allow for variations in soil structure or stratification which often control permeability. Field permeability tests are, therefore, recommended in conjunction with this procedure.

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c. Laboratory Methods

The laboratory constant head permeability test (ASTM Test Method No. D2434) is normally performed on moderate to highly permeable soils and filter materials, while the falling head test using the consolidometer (ASTM Test Method No. D2435) is performed on materials with low permeability. Both tests measure permeability under saturated conditions. For information concerning laboratory methods refer to Appendix D-3.

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d. Field Methods for Design of Basins

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1. Single Ring (Contra Costa County, California)

This test is applicable for infiltration basins in areas with low water table.

A 12-inch (305 mm) diameter or larger steel pipe is driven into the ground a minimum distance of 12-inches (305 mm), with the ground elevation at the time of the test not more than one foot (0.3 m) from the final profile of the bottom of the spreading basin. Water is kept in the test ring for a sufficient period of time to provide calculated saturated infiltration rates under falling head conditions that do not vary by more than 5%. A minimum of three infiltration tests should be made for each basin. For additional test details refer to Appendix D-4.

---

2. Double Concentric Rings

This test is applicable for infiltration basin sites with a low water table. If the permeabilities of the soils under a proposed infiltration basin site vary with depth, tests should be made at sufficient depths to establish the effect of depth on permeability and to aid in determining the required depth of the basin. An infiltrometer is essentially a small model basin consisting of a section of pipe or a bottomless box set to the desired depth in the soil (Figure 5-A-1). The basin is filled to a given depth with water and maintained at a constant head with a float valve for a period of at least a week to measure long-term infiltration rates under saturated conditions. The rate of loss of water in ft/day, in/hour, or cm/day, is defined as the "infiltration rate". If soils are stratified, there is a tendency for the infiltrated water to spread laterally. This will have more effect on infiltration from small basins than from large basins. To compensate for spreading tendencies, a larger outer ring or box is also kept filled with water to form a "buffer zone" to confine the primary flow from the inner test cylinder. Only the flow from the inner ring is used in calculating the "infiltration rate". Refer to Figure 4-A-2, ASTM Test Method D3385 and Appendix D-5.

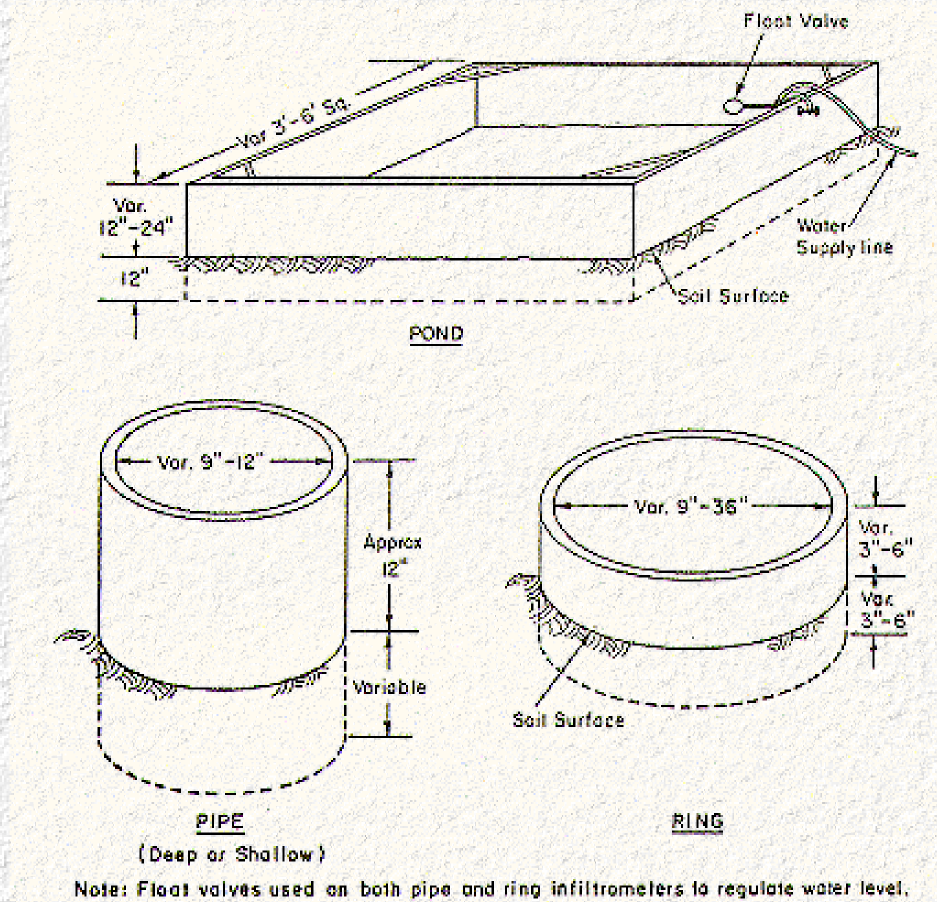
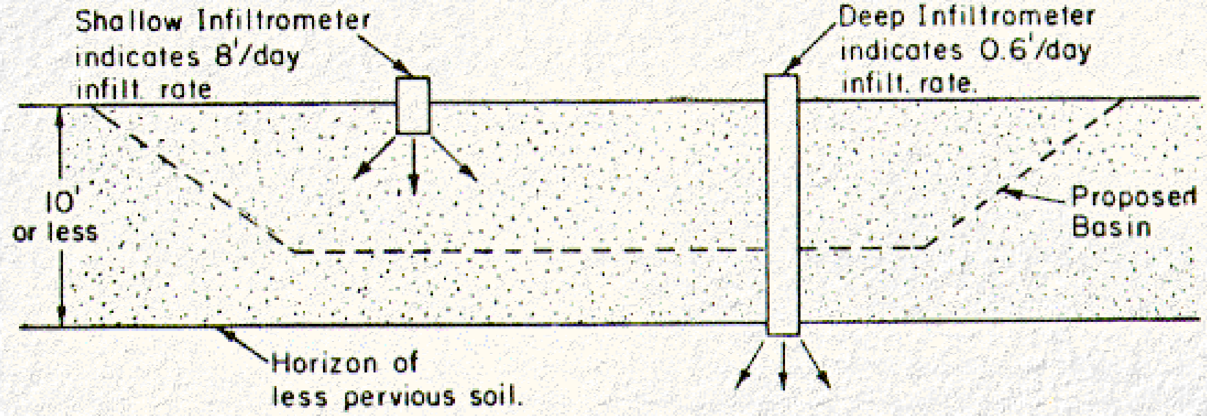
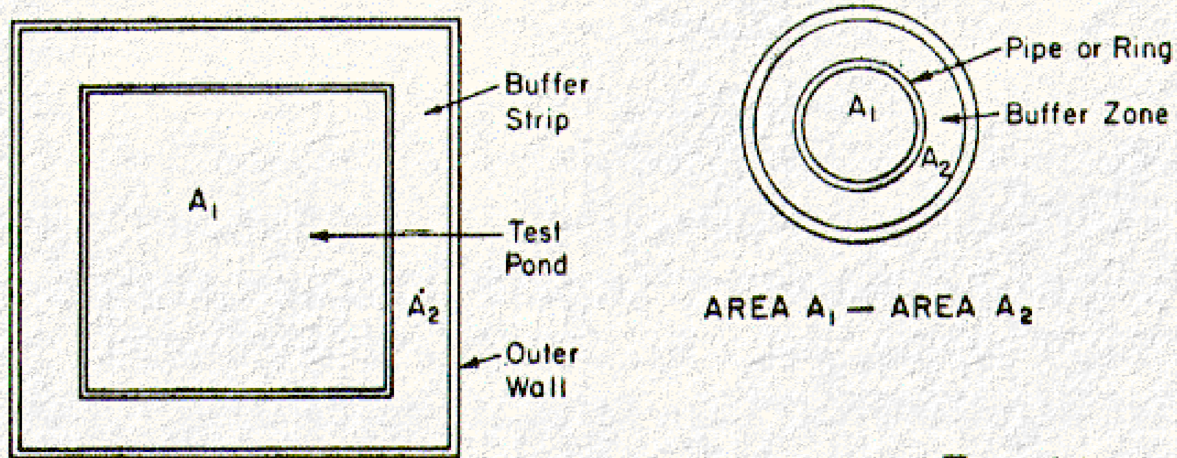
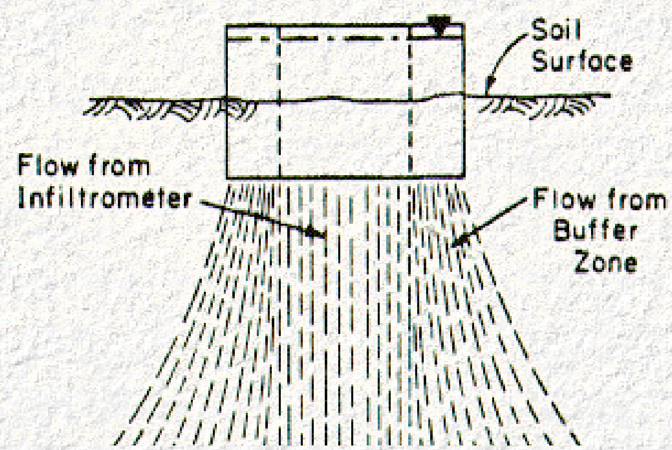


Figure 4-A-1. Infiltrometer Types. ***(Courtesy of Caltrans)***



A. Type of infiltrometer test employed depends on proposed basin depth and thickness of pervious strata. A deep infiltrometer test provides more realistic values when a shallow permeable layer overlays a less pervious one.





B. Buffer zones also provide more accurate infiltration values when a shallow, pervious stratum covers less permeable soil. Flow from buffer zone prevents infiltrometer flow from moving laterally through pervious stratum.

Figure 4-A-2. Infiltrometer Depth and Buffer Zones. *(Courtesy of Caltrans)*

Judgment, based largely on experience, is an Important requirement in evaluating infiltration rate data especially where conditions are non uniform. Robinson and Rohwer in 1957(9) studied infiltration in relation to canal seepage and used a variety of equipment installed in the field. They concluded that large-diameter test rings using 6-feet (1.83 m^3 for an interior ring and 18 feet (5.49 m) for an outer ring provided more accurate measurements than the more commonly used 1 to 2-foot (0.3 to 0.6 m) rings.

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3. Auger Hole Permeability Tests

When water tables are well below the planned bottom elevation of the basin floor, falling head or constant head permeability tests can be performed in auger holes. Numerous procedures are in use for making and interpreting such tests. Methods used by the U.S. Navy are described in Appendix D-6-1. When the U.S. Department of Health, Education, and Welfare method is used for percolation(10), a test hole is kept filled with water for a number of hours, preferably overnight to pre-wet the soil and allow expansive soils to swell at least 24 hours (see Appendix D-6-2 for this procedure). During the test, the drop in water level that occurs in 30 minutes is used as the percolation rate. In sandy or other permeable soils, the time interval between measurements is taken as 10 minutes and the drop that occurs in the final 10 minutes of a 60-minute run is taken as the percolation rate. Basin dimensions can be determined using empirical factors relating basin infiltration to auger hole infiltration, or percolation testing. For design details refer to Section C of Chapter 4,, "Design".

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e. Field Methods for Design of Infiltration Trenches

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1. Falling Head Percolation Tests In Auger Holes (Dade County, Florida)

This test has application for infiltration trenches in areas of high water table. At points located along the centerline of a proposed infiltration trench, holes 9-inches (229 mm) in diameter or larger are bored to at least 2-feet (Q.6 m) below the low-water elevation expected at the site, or to the anticipated elevation of the trench bottom. The portion of the hole below the water table must be kept open during a test. This can be accomplished using a special casing developed specifically for testing in sandy soil as shown in Figure 4-A-3. The special casing is lowered into the auger hole as shown in Figure 4-A-4. The surface elevation, depth to water table. and depth to bottom of casing are recorded. Water is then introduced through the casing until water surface elevation is equal to the design elevation of the top of the proposed drain field which is normally 3 feet (0.915 m) below final ground level. The time is recorded as the water drops in the test hole in 6-inch (152 mm) increments as determined by a float device similar to that shown in Figure 4-A-5.

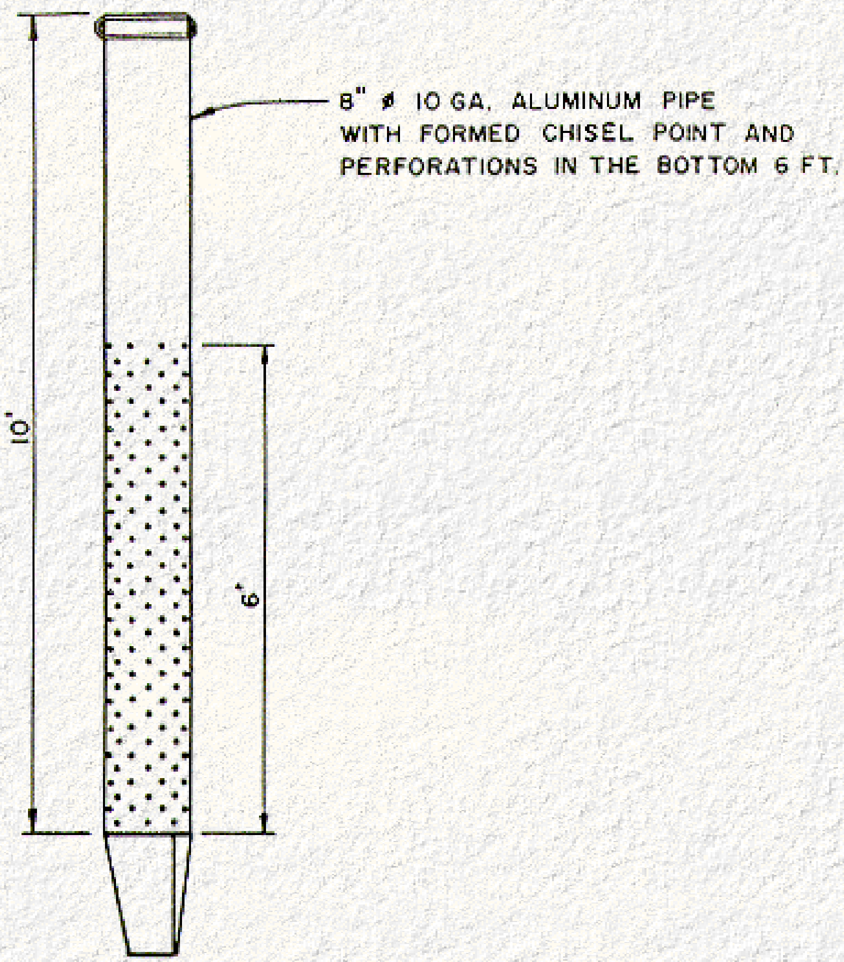

Figure 4-A-3. Casing for Infiltration Test in Sandy Soil. ***(Courtesy of Bristol, Childs & Associates, Coral Gables, Florida)***

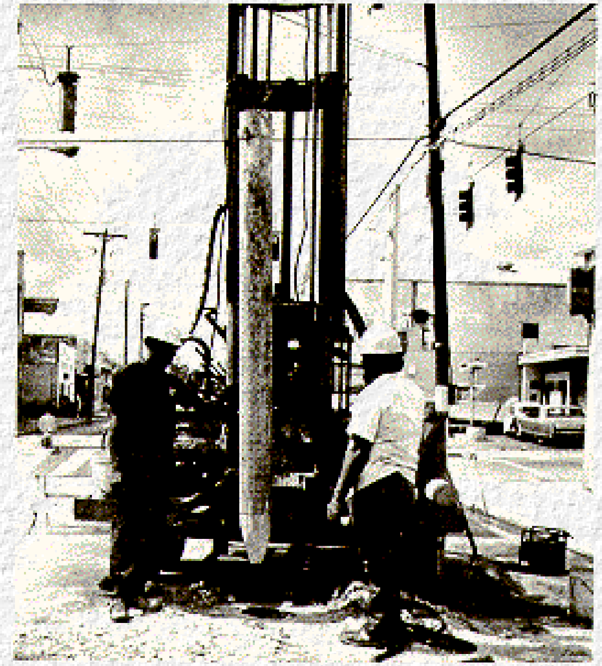

Figure 4-A-4 Special Casing For Auger Hole Permeability Testing. ***(Courtesy of Bristol, Childs & Associates, Coral Gables, Florida)***

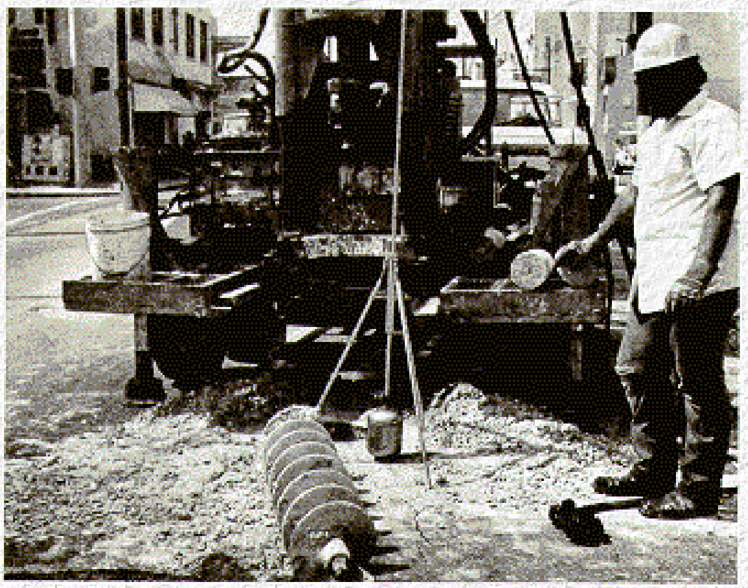

Figure 4-A-5 Special Float Device for Measuring Water Level Change in Auger Holes. ***(Courtesy of Bristol, Childs & Associates, Coral Gables, Florida)***

The volume of water in a specific 6-inch (152 mm) increment of the test hole divided by the time recorded to drop that 6-inch (152 mm) increment results in a rate of infiltration for that specific 6-inch (152 mm) increment:

Q = Infiltration Rate
$Q = V/\Delta t$

where: V = volume of test hole for increment Δh
Δt = time interval for water to fall Increment depth (Δh) as shown in Figure 4-A-6.

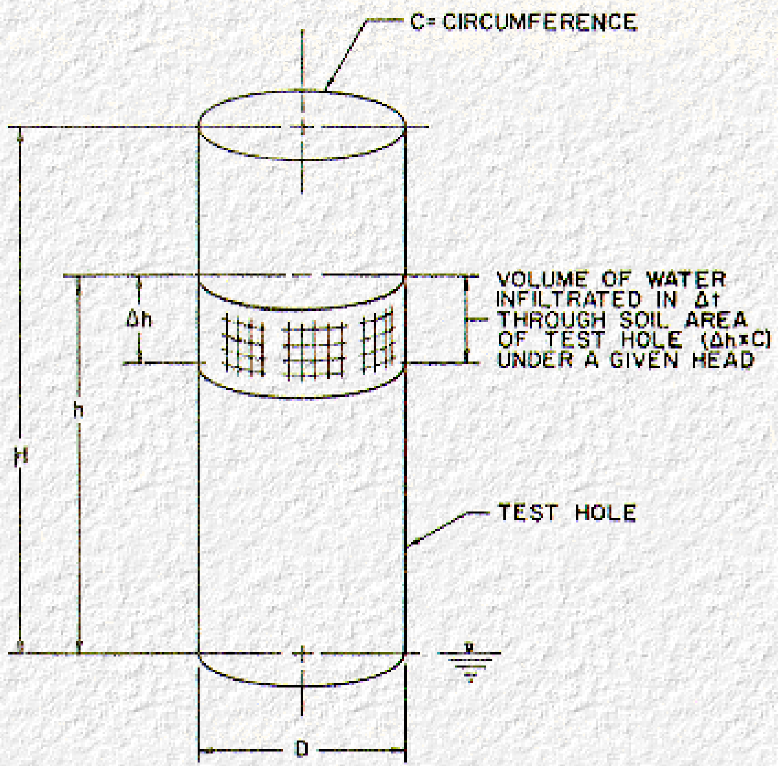

Figure 4-A-6. Test Hole

The rate of infiltration for a specific 6-inch (152 mm) incremental drop divided by the circumference of the test hole gives an infiltration rate for that specific increment per linear foot (0.305 m) of wall area of the test hole as per the following expression:

$Q_{L.F.}$ = Infiltration Rate per lineal foot (0.305 m) of wall

$Q_{L.F.} = Q/C = V/\Delta t x C$

Where: V and Δt are defined above and
C = Circumference of test hole

Since the proposed infiltration trench has two sides, a factor of 2 is applied to give the total exfiltration rate (Q_t) per lineal foot (0.305 m) of trench for a particular 6-inch (152 mm) increment of test hole.

The bottom of the test hole is not considered in the design since it has minimal influence on overall exfiltration rate. In design the bottom of the trench is also ignored as an exfiltration area and provides an added safety factor. The permeability in the lateral direction is usually significantly larger than that in the vertical direction.

Let: Q_t = exfiltration rate per linear foot (0.305 m) of trench (cfs or m^3/sec)

$$Q_t = 2Q_{L.F.} = 2V/\Delta txc$$

Figure 4-A-7 illustrates the exfiltration rate per linear foot (0.305 m) of trench based on percolation tests at 6-inch (152 mm) increments of test hole. The design rate is based on the highest practical elevation of hydraulic head that can be obtained.

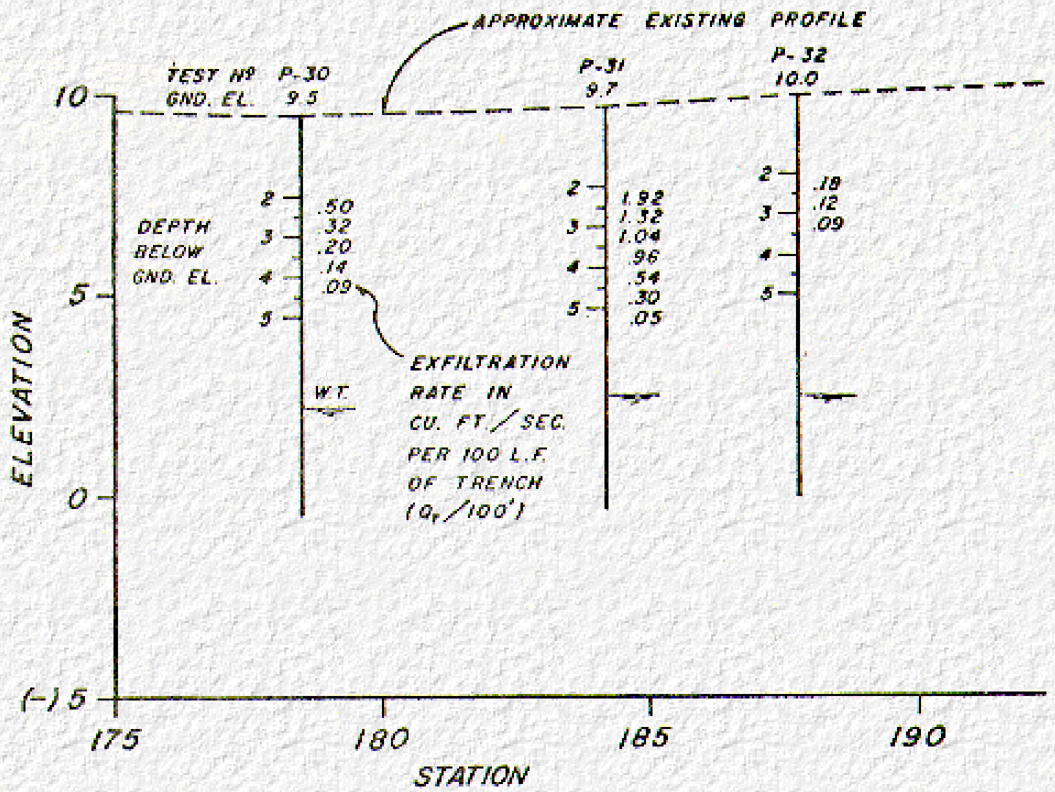

Figure 4-A-7 Exfiltration Rates Determined from Falling Head Percolation Tests *(Courtesy of Bristol, Childs & Associates, Coral Gables, Florida)*

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2. Constant Head Percolation Tests In Auger Holes (Dade County, Florida)

The initial preparation for this test is the same as for the falling head test. However, water is discharged into the test hole at a rate to allow a constant head to be held in intervals of one or more feet (0.3 m or more) depending on depth of hole. This is done to determine if localized soil strata affects infiltration. Water is continually added until the top elevation of the drain field is reached. A constant head should be held for at least 5 minutes at each interval; however, a longer period would provide more accurate infiltration rates.

The infiltration rate per linear foot (0.305 m) of wall area of test hole can be determined for a given constant head using the inflow, Q in cfs or m^3/sec, required to maintain the constant head and relating the flow to the circumference, C, of the test hole, i.e.:

$Q_{L.F.} = Q/C$

The exfiltration rate per linear foot of trench is:

$Q_t = 2\ Q_{L.F.}$

The design rate is based on the highest practical elevation of hydraulic head that can be obtained.

For actual trench design refer to Chapter 4-C.

---

3. Auger Hole Permeability Tests

When water tables are below the planned seepage trenches, falling head (or constant head) permeability tests are frequently made in auger holes drilled to the planned depth of the trenches. Numerous procedures similar to the method described in Section e-1 and Section e-2 are in use for making and interpreting such tests. The tests described in Section d-(3) and in Appendix D-6 can also be utilized for trench design. The trench dimensions are determined using empirical factors relating trench flow with auger hole flow. For details refer to Chapter 4-C.

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f. Field Methods for Design of Wells and Pits

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1. Well Pumping Test

In situations where the flow will be below an existing water table under saturated conditions, well pumping tests provide one of the best methods for estimating in-place permeability. Since the flow to wells is predominately in a horizontal direction (see Appendix D-7), well pumping tests are, in essence, measuring horizontal permeability, which determines the capabilities of underlying soils to discharge seepage laterally. The "well" is pumped while the amount of drawdown is measured in one or more arrays of observation wells. Permeability is calculated as defined in Appendix D-7.

Usually a number of calculations of permeability are made using various combinations of drawdown in pairs of wells and the average is used as representing the permeability of the soil tested.

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2. Auger Hole Permeability Test

Tests similar to those described in Sections d-(3),, e-(1), e-(2) and e-(3) and Appendix D-6 can be utilized to design shallow dry wells and seepage pits.

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3. Theoretical Methods For Estimating Infiltration Rates

The Darcy coefficient of permeability (k) is defined either as the discharge velocity $(V_d = ki)$ under a hydraulic gradient (i) of 1.0, or as the quantity of seepage per unit area under a hydraulic gradient of 1.0. For a given soil under a given state of compaction, etc., k has a specific value that can be used for calculating seepage velocities and seepage quantities under any hydraulic gradient selected for analysis.

In order to apply Darcy's law, or flow nets and other calculation methods using seepage fundamentals, it is necessary to know the Darcy coefficients of permeabilities of the soil formations in which water is flowing. While Darcy's law was originally conceived for saturated flow, it can also be used for unsaturated flow when care is taken to use appropriate coefficients of permeability. The general procedures for using Darcy's law for various cases are presented in Appendix D-8.

Various theoretical methods have been developed for analyzing flow in both saturated and unsaturated soils. A method described by Weaver(5) was developed by the New York Department of Transportation for estimating infiltration rates for unsaturated flow [bottom of basin or trench more than a few feet (1 m ±) above the groundwater level or an impervious stratum]. The method is used for infiltration basins with a large ratio of surface area to perimeter, assuming all outflow is downwards. It provides conservative results for point and line sources (catch basins and trenches) where a large portion of the flow will move laterally through the sides.

Where the bottom of a infiltration basin or trench is below the groundwater table, the infiltration rate should be estimated on the basis of saturated flow. The same is true if the bottom of the basin is only slightly above the groundwater level on an impervious stratum, and the groundwater can be expected to mound up to the bottom of the basin or a perched groundwater table can develop under a basin or trench.

Theoretical considerations in the gravity flow of water out of ditches are given by Musket in 1937(11) and by Harr in 1962(12). Numerous books and reports contain formulas for estimating flow into wells or slots. By making appropriate conversions, these formulas can be adapted to the case of outflow from wells or slots(13).

Approximate two-dimensional methods for estimating flow to large excavations or sumps were given by Cedergren in 1977(14). These methods can also be adapted to the outflow case.

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B. Hydrology

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1. General

The hydrologic input required for the design of any infiltration drainage system is the time-related inflow distribution. This input is usually in the form of a hydrograph or a mass inflow curve. The appropriate hydrologic method used to define this relationship can best be determined by the designer based on consideration of the physical and hydrologic characteristics of the drainage area, the data available, and the degree of sophistication warranted in the design. The designer must be aware of the various methods available to estimate runoff and particularly the limitations of these methods.

It is not the intent of this manual to discuss hydrology in detail nor to recommend a method for estimating runoff. The purpose is rather to discuss data sources, and briefly describe the more commonly utilized runoff estimating procedures and their limitations.

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2. Hydrologic Information

The National Weather Service (NOAA) collects precipitation data and publishes the results in various documents, as listed in Table 4-B-1(1). The information is presented as isohyetal lines on geographic maps of the United States, Puerto Rico, and the Virgin Islands. The technical publications listed under subheadings A and B in Table 4-B-1 give the precipitations to be expected within certain durations and return periods.

Table 4-B-1(1). National Weather Service Publications - Precipitation Data

A. Durations to 1 day and return periods to 100 years		
NOAA Technical Memorandum NWS HYDRO-35 "5 to 60-Minute Precipitation Frequency for Eastern and Central United States", 1977		
Technical Paper 40. 48 contiguous states(l961) (Use for 37 contiguous states east of the 105th meridian for durations of 2 to 24 hours. Use NOAA NWS HYDRO-35 for durations of 1 hour or less.)		
Technical Paper 42. Puerto Rico and Virgin Islands (1961) Technical Paper 43. Hawaii (1962) Technical Paper 47. Alaska (1963)		
NOAA Atlas 2. Precipitation Atlas of the Western United States (1973)		
Vol. I, Montana	Vol. II, Wyoming	Vol. III, Colorado
Vol. IV, New Mexico	Vol. V, Idaho	Vol. VI, Utah
Vol. VII, Nevada	Vol. VIII, Arizona	Vol. IX, Washington
Vol. X, Oregon	Vol. XI, California	
B. Durations from 2 to 10 days and return periods to 100 years		
Technical Paper 49. 48 contiguous states (1964) (Use SCS West Technical Service Center Technical Note -Hydrology Po-6 Rev. 1973, for states covered by NOAA Atlas 2.)		
Technical Paper 51. Hawaii (1965) Technical Paper 52. Alaska (1965) Technical Paper 53. Puerto Rico and Virgin Islands (1965)		

C. Probable maximum precipitation Hydrometeorological Report 33. States east of the 105th meridian (1956) (Use Figure 4-12, NWS map for 6-hour PMP (1975). This map replaces ES-1020 and PMP maps in TP-40** which are based on HM Report 33 and TP-38.) Hydrometeorological Report 36. California (1961) Hydrometeorological Report 39. Hawaii (1963) (PMP maps in TP-43** are based on HM Report 39) Hydrometeorological Report 43. Northwest States (1966) Technical Paper 38. States west of the 105th meridian (1960) Technical Paper 42** Puerto Rico and Virgin Islands (1961) Technical Paper 47** Alaska (1963) Unpublished Reports: *** Thunderstorms, Southwest States (1972) Upper Rio Grande Basin, New Mexico, Colorado (1967)
* National Weather Service (NWS), National Oceanic and Atmospheric Administration (NOM), U.S. Department of Commerce, formerly U.S. Weather Bureau. ** Technical papers listed in both A and C Being replaced by Hydrometeorological Report No. 51 "Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 20,000 Square Miles and Durations from 6 to 72 Hours", available end of 1977. *** Being replaced by Hydrometeorological Report No. 49 "Probable Maximum Precipitation, Colorado and Great Basin Drainages".

Technical Publication No. 40, listed under "A" in Table 4-B-1, is a valuable tool in urban drainage studies, since it give rainfall for various durations and frequencies of recurrence. Other federal agencies such as the USGS and the Corps of Engineers are also good sources of rainfall information. In addition, records are maintained by State Highway or Transportation Departments, State Water Resources Agencies, Cities, Counties, local drainage districts, and utility companies. For rainfall intensity-duration-frequency data for Canada, refer to reference (2) at the end of this chapter.

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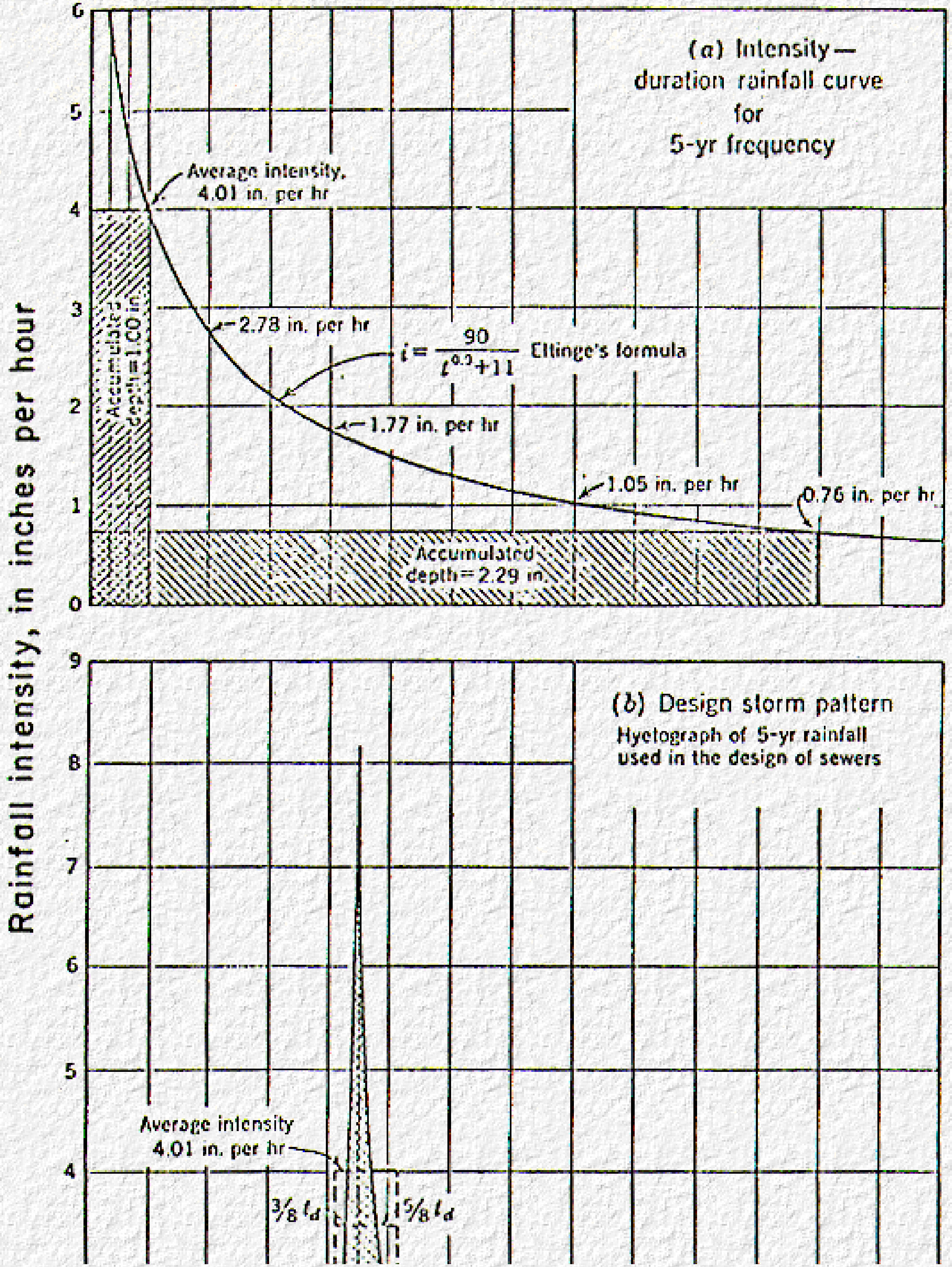
a. Rainfall Intensity - Duration Curves

Rainfall intensity - duration-frequency (I.D.F.) curves are derived from the statistical analysis of rainfall records compiled over a number of years. Each curve represents the intensity-time relationship for a storm of a certain return frequency (3). Refer to Figure 4-B-1a.

The intensity, or the rate of rainfall, is usually expressed in a depth per unit time, with the highest intensities occurring over short time intervals and progressively decreasing as the time intervals increase. The highest intensity for a specific duration for n years of record is called the n year storm, with a frequency of once in n years.

It should be noted that the I.D.F. curves do not represent a rainfall pattern, but are the distribution of the highest intensities over time durations for a storm of n frequency. The rainfall intensity-duration curves are readily available from governmental agencies, and are widely used in the designing of storm drainage facilities and flood flow analysis .



Rainfall intensity, in inches per hour
(a) Intensity—duration rainfall curve for 5-yr frequency
Average intensity, 4.01 in. per hr
Accumulated depth = 1.00 in.
2.78 in. per hr
$i = \frac{90}{t^{0.3}+11}$ Ettinge's formula
1.77 in. per hr
1.05 in. per hr
0.76 in. per hr
Accumulated depth = 2.29 in.
(b) Design storm pattern
Hyetograph of 5-yr rainfall used in the design of sewers
Average intensity 4.01 in. per hr
$3/8\, t_d$
$5/8\, t_d$

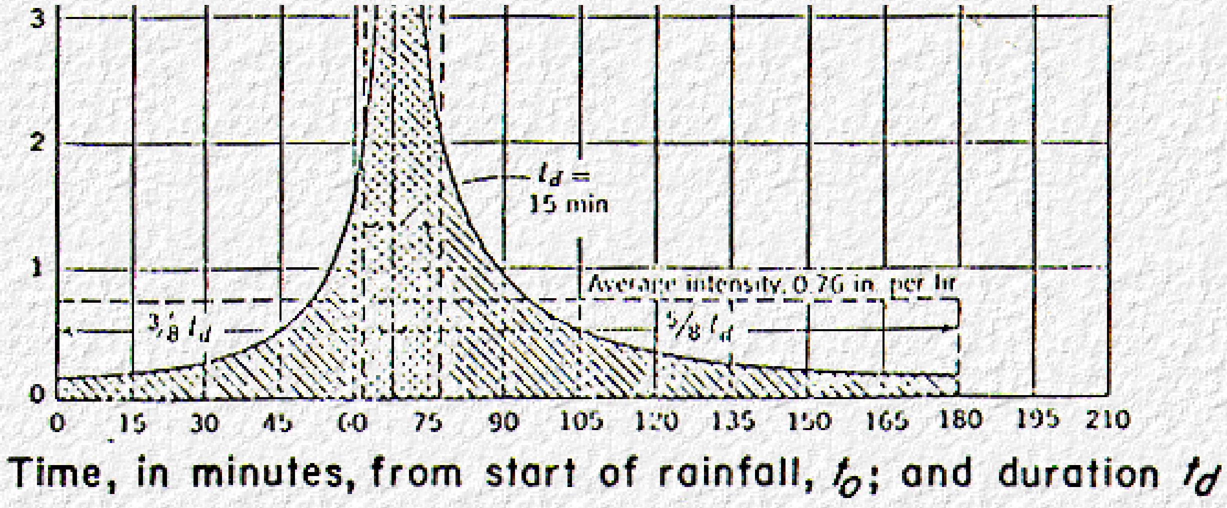


Figure 4-B-1. Intensity-Duration Curve and Concomitant Storm Pattern, Chicago, Ill. (5) & (6)

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b. Rainfall Hyetographs

Rainfall hyetographs are a graphical representation of rainfall over time. Synthetic design hyetographs may be derived from the I.D.F. curve, using the Chicago Method (4).

Briefly stated, this method consists of selecting an allowable storm frequency for the proposed storm drain and determining from rainfall statistics the intensity-duration curve (Figure 4-B-1a) for the selected storm frequency. The chronological storm pattern or hyetoqraph (Figure 4-B-1b) is then determined for storms which are most likely to cause excessive runoff. The design storm pattern or hyetograph is computed to conform at all points of the intensity-duration curve.

The average rate of rainfall during the maximum 15-minute period of the hyetograph equals the rate shown for 15 minutes duration on the intensity-duration curve, and similarly for all other durations (5).

More recently, (1977), the development and use of the non-dimensional triangular hyetograph has been reported by Yen and Chow (6). They report that

> "An analysis of 9,869 rainstorms at four locations indicates that for a given season the non-dimensional triangular hyetographs for heavy rainstorms are nearly identical, having only secondary effects from the duration of rainfall, measurement accuracies of standard U.S. National Weather Service precipitation data, and insignificant effect of geographic locations."

Simple procedures of how to use the non-dimensional triangular hyetograph to produce the design hyetograph are outlined below:

Notation:

D = Depth of rainfall
t_d = Duration of rainfall
T_R = Return period
a = Time to peak

$= t_d a°$ $0.33 < a° < 0.50$

h $=$ Peak rainfall intensity

Procedure:

1. Determine D from NOAA ATLAS
   For the desired storm duration t_d
2. then $h = 2D/t_d$
3. Plot rainfall hyetograph with these parameters (Figure 4-B-2).

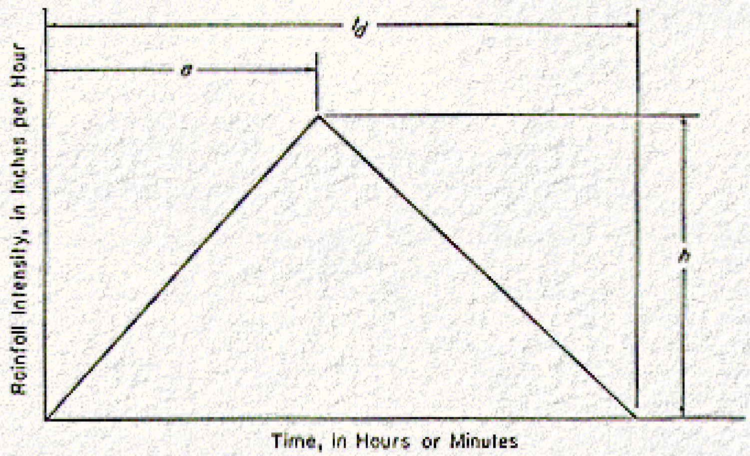

Figure 4-B-2. Simplified Triangular Hyetograph

A simplified hydrograph procedure based on the assumed triangular hyetograph is described in Section 3C(3)..

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3. Methods for Estimating Runoff

There are numerous methods available today for estimating runoff, ranging from the Rational Method developed in 1889 (7) to sophisticated computer simulation models.

The selection of any method must be based on the degree of accuracy required, recognizing the scope and limitations of each method. Except in the rare cases where the infiltration rate of the soils meet or exceed the peak rate of runoff, a graph showing runoff distribution with time must be developed to design an infiltration system. This can be in the form of a hydrograph or a mass inflow curve.

---

a. Rational Method

The Rational Method is widely used to determine peak flows in positive drainage systems by the equation

$$Q = CIA$$

Where Q = Design peak flow (runoff), in cubic feet per second.

C = Coefficient of runoff

I = Average rainfall intensity, in inches per hour for a given frequency and for the duration usually equal to the time of concentration.

A = Drainage area, in acres.

When using the Rational Method, the following assumptions are made:

1. The rainfall intensity is uniform over the entire watershed during the entire storm duration,

2. the maximum runoff rate occurs when the rainfall lasts as long or longer than the time of concentration, and

3. the time of concentration is the time required for the runoff from the most remote part of the watershed to reach the point under design.

---

1. Coefficient of Runoff, C

The only manipulative factor in the Rational Formula is the runoff coefficient C. Judgment should be used in selecting this value, as it must incorporate most of the hydrological abstractions, soil types, antecedent conditions, etc. Typical values for coefficient of runoff are shown in Table 4-B-2 for various types of land use and surface conditions. These coefficients are applicable for storms of 5 to 10-year frequencies. Less frequent higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller affect on runoff (5). It is common practice to select average coefficients and assume that the coefficients will not vary through the duration of the storm. However, it is generally agreed that these coefficients of runoff for any given surface will vary with respect to prior wetting.

Table 4-B-2. Typical Runoff Coefficients for Various Types of Land Use and Surface Conditions (5)

Land Use	Runoff Coefficients (C)
Business:	
Downtown areas	0.70 to 0.95
Neighborhood areas	0.50 to 0.70
Residential:	
Single-family areas	0.30 to 0.50
Multi; units, detached	0.40 to 0.60
Multi units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment dwelling areas	0.50 to 0.70
Industrial:	
Light areas	0.50 to 0.80
Heavy areas	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard areas	0.20 to 0.40
Unimproved areas	0.10 to 0.30
Surface Conditions	

Streets: Asphaltic Concrete Brick	 0.70 to 0.95 0.80 to 0.95 0.70 to 0.85
Drives and walks	0.75 to 0.85
Roofs	0.75 to 0.95
Lawns; Sandy Soil: Flat, 2% Average, 2 to 7% Steep, 7%	 0.05 to 0.10 0.10 to 0.15 0.15 to 0.20
Lawns; Heavy Soil: Flat, 2% Average, 2 to 7% Steep, 7%	 0.13 to 0.17 0.18 to 0.22 0.25 to 0.35

"Usually a substantial period of rainfall will have occurred before the beginning of the time of concentration and consequently, the low coefficients indicated at the beginning of rainfall are in no way representative of storm conditions when the average design intensity occurs." (1)

---

2. Rainfall Intensity, I

The rainfall intensity to be used in the Rational Method for determining peak flow should be for the design frequency, and of a duration equal to the time of concentration. This information is developed as previously discussed.

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3. Time of Concentration, t_c

The time of concentration (t_c) is the time required for runoff to arrive at the point of concentration (such as the inlet to an infiltration system) from the most remote point of the drainage area. Time of concentration is generally developed relative to the initial point of concentration. Drainage system calculations also require the addition of time of flow in the system between the inlet and the point of control. Inlet times generally used in urban drainage design vary from 5 to 20 minutes with the channel flow time being determined from pipe flow equations.

---

4. Limitations of Rational Method

The Rational Method does have limitations and should only be applied to relatively small drainage areas. The maximum acceptable size of the watershed varies from 200 to 500 acres (0.90 to 1.422 Km^2) depending upon the degree of urbanization. The APWA Special Report No. 43 (8) recommends that urban drainage areas should be limited to less than 20 acres (0.284 Km^2) in size, such as rooftops and parking lots. As drainage areas become larger and more complex, the C coefficient cannot account for the many natural hydrological abstractions, surface routing, and antecedent moisture conditions.

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b. Modified Rational Method for Development of Mass Inflow Curves

The Rational Method has been used to calculate the total cumulative volume of rainfall runoff versus time (mass flow) by modifying the formula to read V = CIAT.

Where

V = Volume of runoff in cubic feet
C = Coefficient of runoff
I = Average rainfall intensity, in inches per hour for a given frequency and for selected durations of time in increments sufficient to plot a curve showing total cumulative volume of rainfall runoff versus time
A = Drainage area, in acres
T = Time in seconds which corresponds to the selected durations of rainfall.

The following is an example calculation for the mass flow curve for a 3-year frequency design storm, using hourly intensities from Figure 4-B-3 and the Modified Rational Equation:

Assume A = 1.0 acre (4,047 m^2) drainage area and C = 0.9. Using the Modified Rational Formula,
V = CIAT, CA = (0.9) (1.0) = 0.9. The following cumulative volumes of flow are developed:

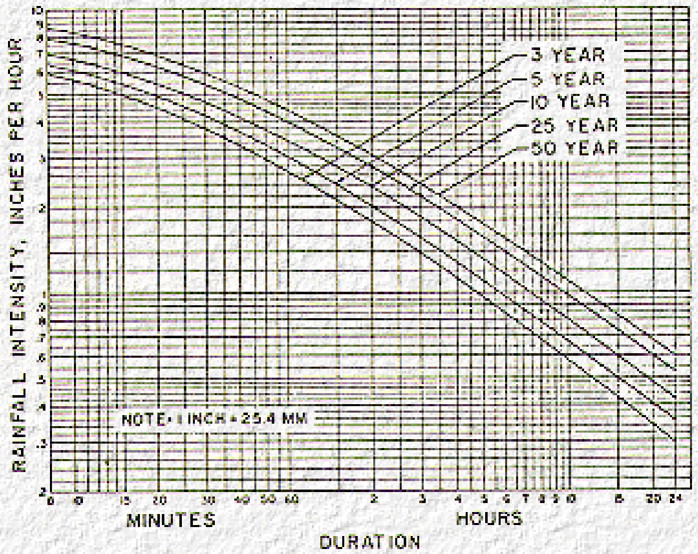

Figure 4-B-3. Rainfall Intensity-Duration-Frequency Curves for Zone 5, Miami. ***(From Miami DOT)***

Time Minutes	CA	x	I Inches/hr.	x	Time Seconds		Volume Cu. Ft.
10	0.9	x	5.60	x	600	=	3,024
15	0.9	x	4.90	x	900	=	3,969
20	0.9	x	4.40	x	1,200	=	4,752
30	0.9	x	3.75	x	1,800	=	6,075
60	0.9	x	2.65	x	3,600	=	8,586
90	0.9	x	2.10	x	5,400	=	10,206
120	0.9	x	1.75	x	7,200	=	11,340

150	0.9	x	1.50	x	9,000	=	12,150
180	0.9	x	1.35	x	10,800	=	13,122
240	0.9	x	1.10	x	14,400	=	14,256
360	0.9	x	0.83	x	21,600	=	16,135

The resulting inflow curve is shown in Figure 4-B-4. Specific applications are discussed under "Design of Storm Water Collection and Disposal Systems", in Chapter 4-C.

It should be recognized that most mass inflow curves constructed using the above procedure do not truly reflect the expected accumulated runoff as a function of time, since the probable storm pattern and the storage affects of the watershed are not considered. However, the results in sizing underground disposal systems using this procedure should be conservative in most instances. The simplicity of the method makes it attractive where more detailed studies may not be warranted.

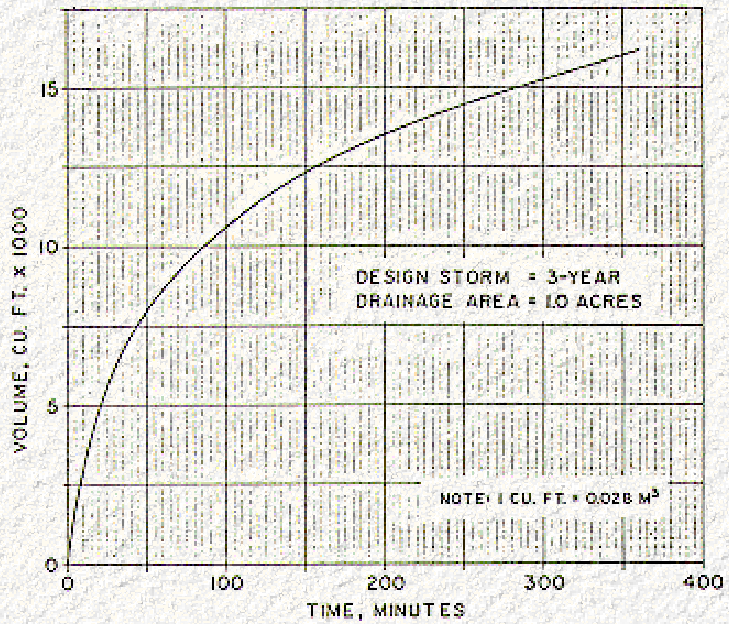

Figure 4-B-4 Mass Onflow Curve. ***(Courtesy of Bristol, Childs & Assoc. Coral Gables, Florida)***

---

c. Hydrograph Methods

As previously indicated in this chapter, hydrograph methods relate runoff rates to time during a design storm, and are generally more applicable to larger watersheds, though used also with small watersheds, particularly where storage is considered.

Natural hydrographs are those obtained directly from the flow records of a gaged stream channel or conduit. Synthetic hydrographs are developed using watershed parameters and storm characteristics to simulate a natural hydrograph. A unit hydrograph is defined as a hydrograph of a direct runoff resulting from 1 inch (25.4 mm) of effective rainfall generated uniformly over the watershed area during a specified period of time or duration. The unit hydrograph can be used to develop the hydrograph of runoff for any quantity of effective rainfall.

The unit hydrograph theory, assumptions, and limitations are discussed In detail in references (9) and (10).

1. Synthetic Unit Hydrographs

In most drainage basins rainfall runoff data from which unit hydrographs can be derived is unavailable, thus a synthetic unit hydrograph must be derived. The U.S. Soil Conservation Service (SCS) has developed a method of hydrograph synthesis which is now being widely used.

The development of the SCS unit hydrograph technique is well documented (11). Studies by the U.S. Soil Conservation Service over the last 30 to 35 years have resulted in empirical relationships between rainfall runoff and the associate land use which are used in conjunction with the SCS Unit Hydrograph Method. Each particular land use is assigned a corresponding runoff curve number (CN), which is an indication of the runoff potential. The value is based on a combination of hydrological soil group, treatment class and antecedent conditions.

The following are limitations of SCS Unit Hydrographs:

1. The drainage area should be limited to 20 square miles (51.8 Km^2). If the total watershed is very large, it should be broken down into uniformly shaped divisions with a maximum of 20 square miles (51.8 Km^2) each.
2. The drainage areas should have a constant CN value.
3. There should be a homogeneous drainage pattern within the drainage area.
4. Care should be taken in determining the representative CN value as it will have a direct effect in the hydrograph peak.

---

2. SCS Tabular Hydrograph Method

This method provides a tabular approach to estimating peak concentration and travel time. It also develops hydrographs for each sub-drainage area and then routes them through the watershed area resulting in a composite hydrograph at the outfall. This method can readily predict the increase in peak flow when all or a portion of the watershed is to be developed. The SCS tabular method is described along with examples of applications in SCS Technical Release No. 55 (12).

---

3. Simplified Equivalent Triangular Hydrograph.

A normal curvilinear hydrograph can usually be represented by an equivalent triangle as shown in Figure 4-B-5. Both graphs represent the same amount of runoff and the game time to peak; therefore, for practical purposes the triangle is an adequate representation of the curvilinear graph.

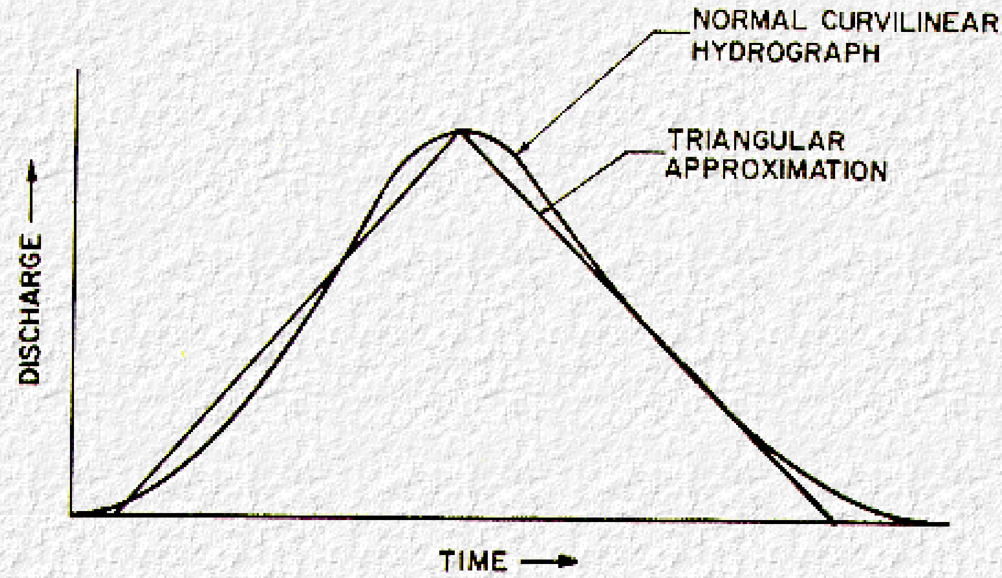

Figure 4-B-5. Triangular Approximation of Runoff Hydrograph

The simplified hydrograph procedure is based on an assumed triangular hyetograph as previously described in this chapter. Abstractions are applied from information using SCS curve numbers or guidance provided by local experience.

Assuming a linear watershed response (i.e., the area contributing to runoff increases more or less uniformly up to the time of concentration), a triangular distribution of excess rainfall may be converted to an approximate triangular runoff hydrograph as shown in Figure 4-B-6. The peak runoff is equal to the maximum average effective rainfall intensity over the time of concentration and is shifted to the right (1 - a°) t_c units.

The peak runoff in cfs (m^3/sec) is determined from the equation:

$$Q_p = I_p (1-t_c/2b) A$$

Where I_p = Maximum effective rainfall intensity in inches/ hour (mm/hr.)

A = Area of the drainage basin in acres

t_c = Time of concentration in minutes, and

b = Duration of effective rainfall in minutes ($b \geq t_c$).

The lengthening of the time base, Δ, is given by

$$\Delta = \frac{t_c}{(2b - t_c)} b$$

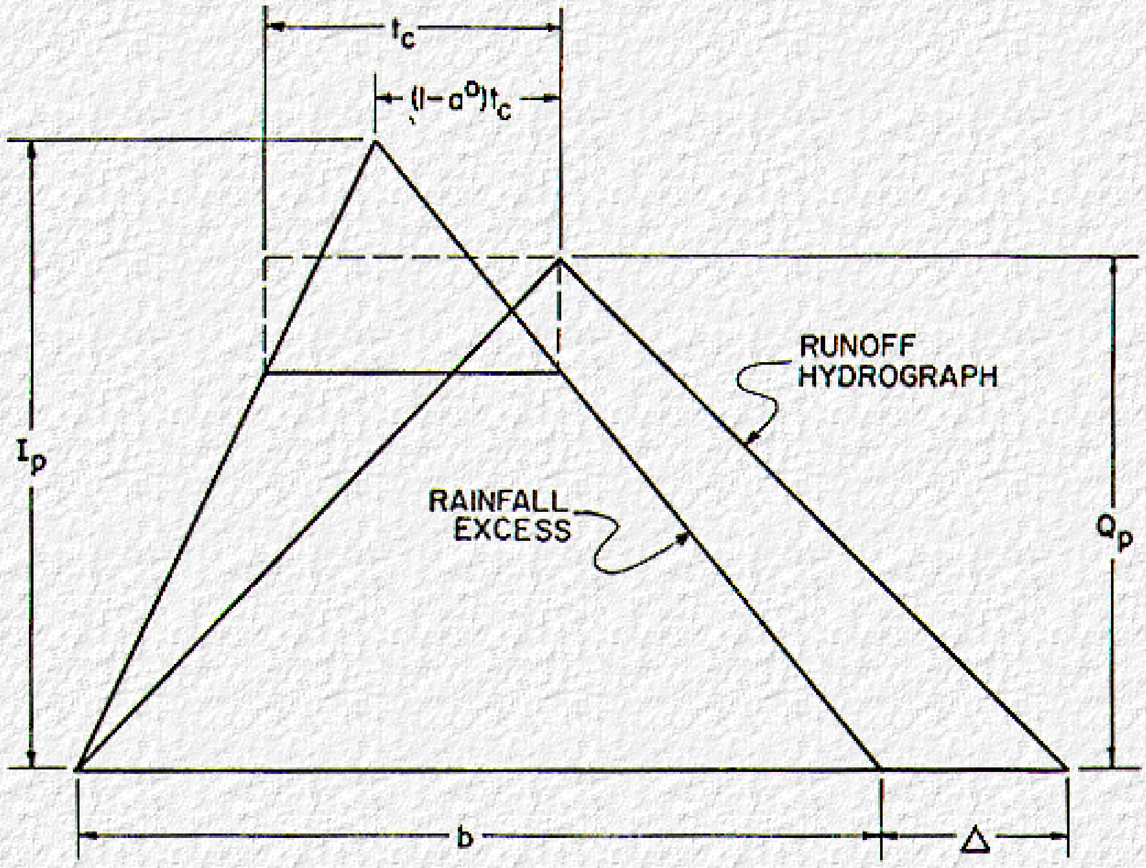

Figure 4-B-6. Relationship between Simplified Triangular Plot of Rainfall Excess and Triangular Runoff Hydrograph

Example

Assume the following:

Drainage area = 6 acres (0.156 Km^2)
Design Storm Frequency (Return Period) = 10 years
Duration of Storm = 2 hours
a° = 0.33
Rainfall Depth, D = 3.63 inches (92.2mm)
Time of Concentration, t_c = 30 minutes
Infiltration rate of drainage area:
Initial = 1 inch/hr. (25.4mm/hr.)
Final = 1/4 inch/hr. (6.4mm/hr.)

Step 1 Derive and plot the triangular hyetograph as previously described (Figure 4-B-2).

$$h = \frac{2D}{t_d} = \frac{2(3.63)}{2} = 3.63\ \text{inches/hr.}\ (92.2\ \text{mm/hr.})$$

$$a = t_d a^{\circ} = 2\ \text{hr.}\ (60\ \text{min./hr.})(0.33) = 40\ \text{minutes}$$

- Step 2 Deduct losses as shown in Figure 4-B-7. (The resulting shape must be approximately a triangle.)

- Step 3 Scale the new time base and the maximum effective rainfall intensity.

  b=105 minutes

  I_p=2.9 inches/hr. (73.7 mm/hr)

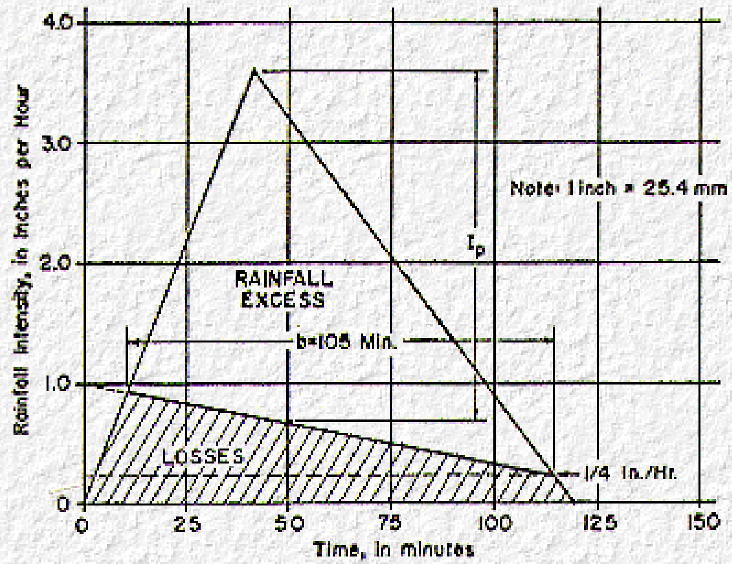

Figure 4-B-7. Triangular Hyetograph Showing Peak Rainfall Intensity

- Step 4 Compute Q_p, and the time to peak.

$$Q_p = I_p\left(1 - \frac{t_c}{2b}\right)A$$

$$= 2.9\left(1 - \frac{30}{210}\right)6 = 14.9\ \text{cfs}\ \left(0.417\ \text{m}^3/\text{sec}\right)$$

$$= \left(\frac{t_c}{2b - t_c}\right)b = \left(\frac{30}{210 - 30}\right)105 = 17.5\ \text{minutes}$$

Time to peak = 40 + (1-0.33)(30)

= 60 minutes

Step 5 Plot the triangular runoff hydrograph using these parameters (Figure 4-B-8a).

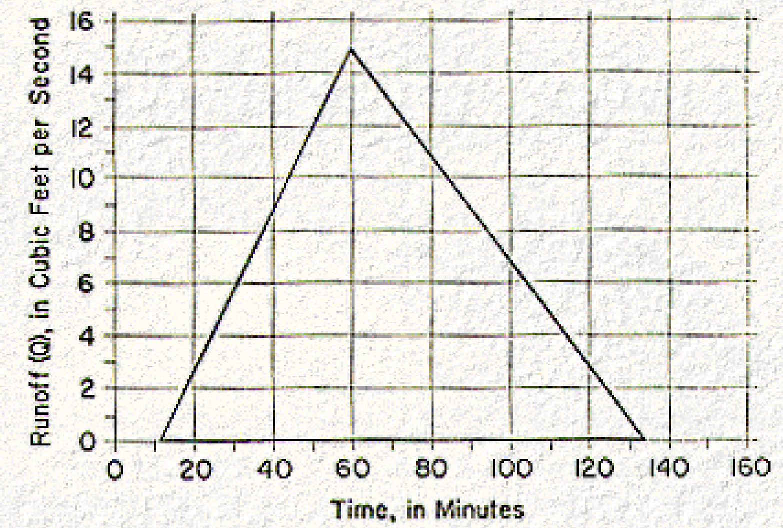

Figure 4-B-8a. Triangular Runoff Hydrograph

Step 6 The cumulative runoff curve is determined by summing the area under the triangular hydrograph from left to right and plotting the results as a function of time (Figure 4-B-8b).

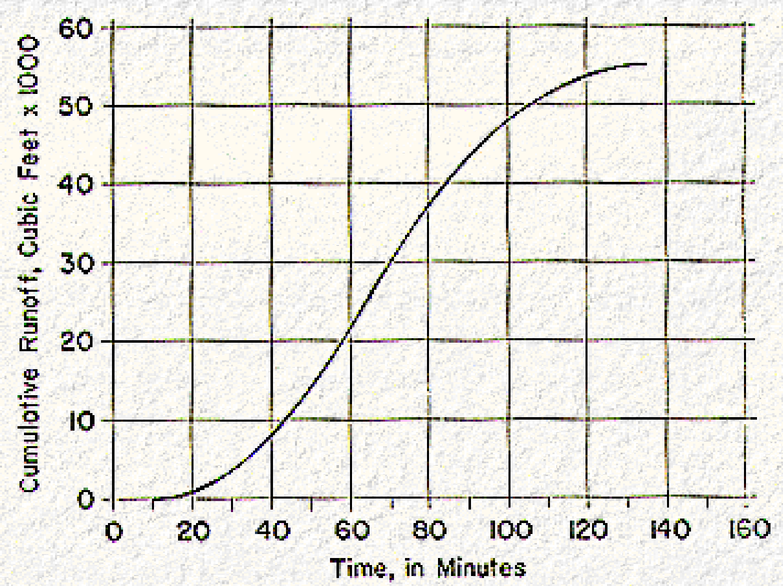

Figure 4-B-8b. Cumulative Runoff Curve

Runoff hydrographs will differ depending on the storm duration chosen. The designer may need to investigate various types of storms in sizing an underground disposal system.

---

d. Computer Modeling

In recent years computer models have been developed to aid the designer in his analysis of the hydrological and hydraulic analyses of drainage systems. Of the numerous models available today, the ones listed below are believed to be most applicable in the generation of runoff for the design of subsurface disposal facilities:

SWMM: A sophisticated hydrologic and hydraulic simulation model used primarily for complex urban drainage systems.

ILLUDAS: A simulation model with the capacity of accurately simulating the runoff from urban areas, but continuing a relatively simple routing procedure for pipe flow.

HYMO: A model well-suited for generating runoff from rural or undeveloped lands (may also be used in urban areas) based on the SCS CN runoff parameters, but with a modified unit hydrograph procedure.

---

4. Summary

This chapter has provided a brief overview on the hydrology involved in estimating storm water runoff for underground disposal systems. A reference list is provided at the end of this section to allow the designer to obtain additional Information on methods and techniques which he feels are applicable to his study area. Particular design applications are contained in Chapter 4-C.

Table 4-B-3 summarizes the characteristics and application of the methods covered in this chapter, to assist the designer in the selection of the appropriate method.

Table 4-B-3. Runoff Models

Method	Drainage Area	Required Information	Variables	Output	Applications
Rational Method	<20 acres (APWA) Spec. Rep. No. 43 <500 acres (FHWA)	Land Cover Time of Concentration IDF Curves	Runoff Coefficient (C)	Peak Flows	Minor and Major Storm System Design
Unit Hydrograph		Rainfall		Hydrograph	Flood Flows
SCS Unit Hydrograph	Up to 20 sq. Mi. if large water-shed. break down to 20 sq. mi .	Time of Concentration	Runoff Curve No. (CN) Runoff (Q) inches >1. 5 CN >60	Hydrograph	Minor and Major Storm Flood Flow Systems Storage Volumes

SCS Tabular Method	Up to 20 sq. Mi. if large water-shed. break down to 20 sq. mi.	Soil Type 24 Hr. Cumulative Rainfall Time of Concentration	Runoff Curve No. (CN) Accounts for Hydrological Abstractions	Hydrograph	Flood Flows Major Storm System Storage Volumes
SCS Graphical Method	<20 sq. Mi.	Type Cumulative Rainfall	Runoff Curve No. (CN) Runoff (Q) inches >1. 5 CN >60	Peak Flow	Flood Peaks Minor and Major Storm Systems
Computer Modeling	Dependent on capacity of program	See Users Manual	See Users Manual	Hydrographs	Trouble Shooting Design of Minor and Major Storm Systems Storage Vol umes
Rational Mass Inflow	<20 Acres Spec. Rep. #43	Landcover IDF Curves	Runoff Coefficient (C)	Storage Volume	Detention and Infiltration Facility Design
Simplified Equivalent Triangular Hydrograph	<200 Acres	Soil Type Rainfall Hyetograph Time of Concentration	Runoff Curve Runoff No. (CN) (Q) inches > 1.5 CN>60	Hydrograph	Small Storage and Infiltration Facility Design

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C. Design of Storm Water Collection and Disposal Systems

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1. Methods of Collecting Storm Water

Surface runoff can be collected at either a point or along a linear collector. The point collector can consist of a catch basin, inlet, small pond, or basin. A linear collector can be a scale, ditch, curb and gutter, or perforated or slotted pipe.

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2. Methods of Disposal of Collected Storm Water

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a. Positive Systems

Any system that conveys accumulated runoff directly to a stream, canal, river, lake, sea, or ocean is considered a positive system. These would include normal outfall systems such as underground pipes, box culverts, and open or covered trenches or ditches. Pipe sizing and design specifics of such systems are not within the scope of this manual. However, for detailed information refer to a hydraulics textbook or agency publication on storm drainage. A few of many references available on the subject are listed at the end of Chapter 4-B.

---

b. Infiltration Systems

There are three basic types of infiltration systems: basins, vertical wells or pits, and trenches. Each has a particular "best area of use", dependent upon situation and conditions.

---

1. Basins

Basins are generally open excavated depressions of varying size. They can be located in excess land areas within highway rights-of-way or within non-used land areas of residential developments. Designed for storage and infiltration, basins can be a practical and economical means of disposing of highway or subdivision surface water. Their single drawback is that they require considerable space. However, where space is available, they are the least expensive recharge system to construct per unit of water handled.

---

2. Trenches

Trenches can be of either the open type, such as ditches or stales; or covered, with concrete, steel, or aluminum lids if the trench walls are self-supporting.

Trenches are ideal for use within right-of-ways that afford only limited space. They can be placed in complex alignments to suit almost any parking lot or landscaped areas; and are ideal for use in almost any type of permeable soil. Trench designs for three different soil and geologic conditions are discussed below.

---

a. Trenches in Rock

The trench in permeable rock is the least expensive to construct; however, the following conditions must be met:

1. The rock must be able to support a specified wheel load on a covering concrete slab or other suitable cover.
2. The rock must be amenable to excavation without blasting, or the cost becomes prohibitive.

The inlet to the system can be placed directly over the slab cover, with discharge directly into the trench. A more acceptable method is to set the inlet and catch basin adjacent to the trench and pipe the inflow to the trench. This lessens the introduction of debris into the system. With this type of trench, manhole access must be provided to facilitate cleaning and inspection.

---

b. Trenches in Stable Soil

In this type of trench, perforated or slotted pipe is used as the conduit. Coarse aggregate between the pipe and trench wall prevents wall side collapse and distributes collected water to the trench walls.

The trench is usually 4 to 5-feet (1.2 to 1.5 m) in width and of sufficient depth to reach a permeable soil layer or the water table.

Coarse aggregate or other free-draining material is generally placed in the bottom of the trench and brought up to a specified pipe flowline grade, generally a minimum of 2 feet (0.6 m). Perforated or slotted pipe is then placed in the trench and the trench is backfilled with the coarse aggregate to a design elevation. A 6-inch (0.15 m) thickness of filter material is placed over the aggregate backfill and covered with a barrier consisting of building felt, tarpaper, or other suitable material to prevent piping and possible surface subsidence. The trench is then backfilled with native soil and pavement similar to Figure 2-4 in Chapter 2.

---

c. Trenches in Cohesionless Soil or Sand

Although trenches in cohesionless soil require a different type of construction, the design, final shape, and size are the same as for a trench in stable soil. However, side slopes of 1-1/2:1 or 2:1 may be required, if the walls are not shored during construction. It is recommended that filter cloth be used along the periphery of the trench to prevent migration of soil fines into the coarse aggregate backfill.

In a trench system where perforated pipe is used, a non-perforated section some 6 to 8 feet (1.83 to 2.44 m) is used to connect the trench to the catch basin or inlet. This serves to prevent piping along structure and subsidence. A concrete slab is generally placed around the catch basin or inlet.

In the design of a trench system, any one of the above types or combination thereof, may be used. Wells may be placed in the bottom of trenches that are marginal in their infiltration ability. It is recommended that a positive overflow pipe also be provided to allow for the unusual storm where possible.

---

3. Vertical Wells or Pits

Either pressure or gravity type wells may be utilized. Pressure or injection wells are mostly used in sewage effluent disposal, although they could be used in special circumstances for storm water disposal.

Gravity wells are the type most commonly used for storm water disposal. They may be employed independently or in conjunction with a trench or basin system. Where used alone, they are intended to infiltrate the drainage from small areas. Wells are sometimes used in conjunction with another infiltration system to penetrate an impervious layer that prevents or hinders the necessary percolation. Shallow wells may extend down to a depth of 25 feet (7.63 m). Wells whose depth exceeds 25 feet (7.63 m) are generally considered deep wells. Catch basin covers may be placed directly over wells in rocky areas. In non-rocky areas, a perforated or slotted pipe is used with uniformly graded aggregates backfill to help disperse water and reduce velocity to prevent soil erosion.

---

c. Retention or Storage Systems

These systems are generally constructed in soils of low permeability where storm water storage is the main criterion and infiltration is a safety factor in the design. Pipes, trenches, basins, wells, and reservoirs all serve this function to some degree. Outflow is through slow soil infiltration and evaporation.

---

d. Detention Systems

Detention systems are similar to retention or storage systems except that storm water is held for later release through a normal outfall. Refer to Figure 4-C-1. These systems are designed to retain runoff during heavy periods of rainfall where runoff would overtax existing drainage systems and cause downstream flooding. Surface systems are used to provide partial treatment of water pollutants through oxidation.

Flow Regulators

Devices are available for installation in storm drainage systems to control the rate of flow into existing sewers that have limited capacity. The regulator is placed between a storage reservoir and the sewer to release a predetermined rate of flow. These systems should not be subject to clogging or variations in hydraulic head. Various versions are also available for permitting roof storage for later release through roof drains. The placement of these devices is illustrated in Figure 4-C-1.

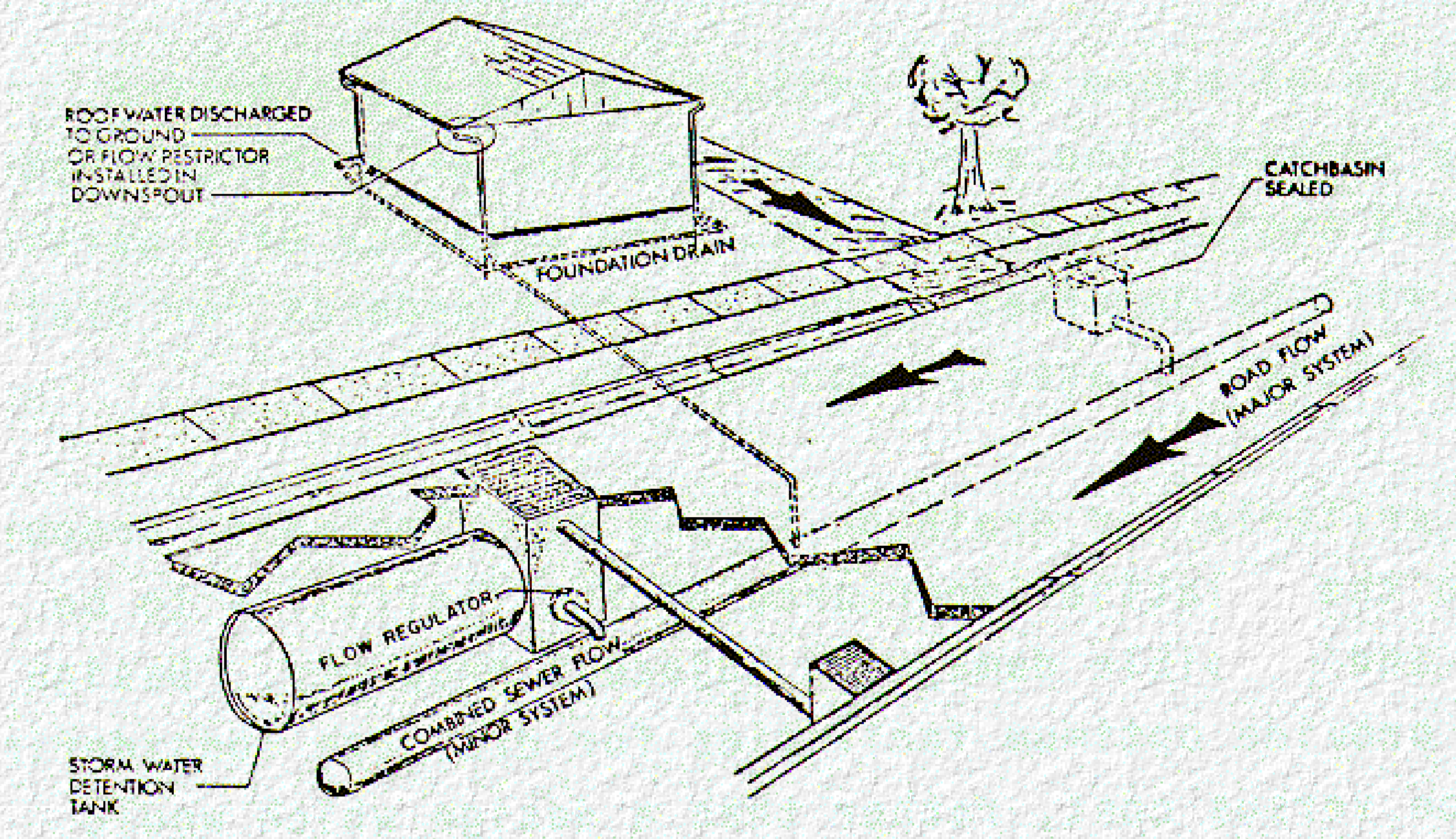

Figure 4-C-1 Inlet Control Method with Detention. ***(Courtesy of Paul Theil Associates Limited, Bramalea, Ontario, Canada)***

---

e. Combination Systems

These systems consist of a combination of the above systems serving as smaller subsystems and they may also include both positive drainage and exfiltration or infiltration drainage components.

Figure 4-C-2A shows the use of a surface detention pond for storing and infiltrating storm water prior to discharge into a storm drain system. The system could also incorporate exfiltration features.

Figure 4-C-2B illustrates the use of a large diameter perforated in-line detention pipe which provides both temporary storage and infiltrates water into soil.

---

3. Design Requirements

The design of storm water collection and distribution systems requires the proper application of technical data and sound engineering judgment. The adequacy of a particular system will be governed by the design standards for drainage based on local or agency requirements.

Design criteria for drainage should be selected in proportion to the relative importance of the facilities to be served and possible damage to adjacent property. Design should provide optimum facilities for drainage considering function versus cost, rather than a design which would meet an arbitrary minimum standard.

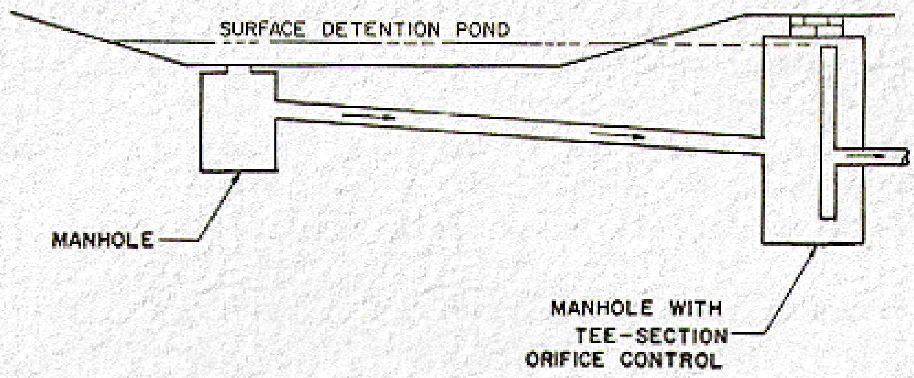

Figure 4-C-2A. Surface Storage and Detention Prior to Discharge in Storm Drain System. ***(Courtesy of Paul Theil Associates Limited, Bramalea, Ontario, Canada)***

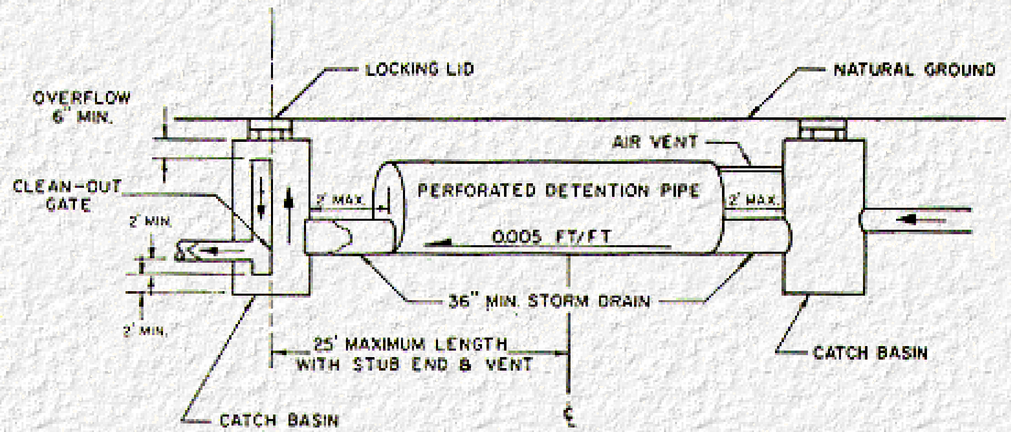

Figure 4-C-2B. Underground Detention-Exfiltration Feature of Storm Sewer System. ***(Courtesy of Paul Theil Associates Limited, Bramalea, Ontario, Canada)***

---

a. Positive Systems

Many agencies publish hydraulic capacities, which permits the direct selection of pipe or channel size for design. A particular design could incorporate the use of infiltration elements to reduce pipe size requirements for the positive outfall portion of the system.

Although this text does not present guidelines for the design of positive storm drain systems, various references for such design are provided at the end of Chapter 4-B, "Hydrology". The primary emphasis of this manual is the presentation of design guidelines for infiltration systems as discussed below.

---

b. Infiltration Systems

---

1. General Considerations

. Perform necessary surveys and investigations to cover evaluation of alternative disposal systems, environmental and legal considerations, soils exploration, and relative costs for feasibility of each alternative. (Refer to Chapter 3-A, Chapter 3-B, and Chapter 3-C.)

b. Determine the infiltration rates of local soils using procedures defined in Chapter 4-A.

c. Develop a discharge time relationship for watershed area.

d. Consider on-site detention of stormwater runoff in the design of drainage facilities. Possible detention locations include:
   1. Parking areas
   2. Rooftops
   3. Excess street and road rights-of-way
   4. Excess land within interchanges
   5. Green belt areas
   6. Existing ponds and lakes

The use of the above areas for this purpose will be governed by local regulations.

---

2. Basins

---

a. Design Considerations

The most important consideration in infiltration basin design is the storage volume required. The basin must have sufficient capacity to contain all the runoff from the area it drains during a specified storm frequency. The infiltration rate for the soils of a particular site should be evaluated by methods such as those described in Chapter 4-A. The infiltration rate can be related to various hydraulic head conditions and can easily be equated to cubic feet per second per square foot (or $m^3/sec/m^2$) of infiltration basin for determination of final storage volume.

The basin should be designed to provide a factor of safety. The factor of safety should be developed taking into consideration the risk to life and property. Some examples of approaches used by different agencies are illustrated further in this chapter.

Basin shape depends largely on the configuration of the available site. The shape providing the greatest side area will infiltrate water most rapidly. An increase in basin floor area may not bring about a corresponding increase in the rate of infiltration. Lateral drainage through the basin wall is generally several times more rapid than vertical percolation. Also, the bottom becomes less pervious as silt deposits build up.

Increased depth will provide greater lateral infiltration area and a higher effective head to the permeable strata. Basin sideslopes should be 2:1 or flatter to prevent erosion and improve appearance. Installations in residential areas should be fenced.

Detention ponds or sedimentation basins can be used in conjunction with infiltration basins so that suspended solids will settle out before the water is released into the infiltration basin. These ponds must be large enough to hold storm runoff for a sufficient settlement time, which could vary from one or two hours to a day or more depending on water quality. Detention ponds are usually larger and shallower than the actual infiltration basin. For design of detention ponds and sedimentation basins refer to American Public Works Association Special Report No. 43(1).

Landscaping of infiltration basin facilities ties creates pleasing appearance and should always be considered when basins are located near residential areas. Without landscaping and maintenance, a basin will accumulate the inevitable old tires, broken glass, and trash thus becoming a community nuisance. The landscaped basin can be used as a park or recreation area. Such a project requires plants, trees, and facilities capable of withstanding temporary inundation. Buildings should be located on high ground and basin sides gently sloped to give a park-like appearance.

A typical park-type installation is shown in Figure 4-C-3. It consists of a large detention and sedimentation basin with a smaller basin for infiltration of storm water.

During summer months, when the grass in these park basins must be sprinkled and additional inflow may occur from domestic watering or irrigation, some water may stand at the low points in the basin. A central drain should be installed to infiltrate water into the soil by way of infiltration trenches or vertical wells. Trenches of this type are usually 4-12 feet (1.22 to 3.66 m) wide and at least 6 feet (1.83 m) deep and backfilled with a filter material consisting of gravel or sand. These trenches and wells are used to penetrate low-permeability strata near the surface in order to drain standing water rapidly. Typical installations are shown on Figure 4-C-4.

Filter material should be graded according to the standard criteria as defined in this chapter and should be mounded over the top of the backfilled trench to a depth of approximately two feet (0.6 m).

The filter layer will prevent the pervious backfill material from becoming clogged if it is designed to prevent the intrusion of the silt-sized particles suspended in the storm runoff water. Sand drains or drain wells are shallow, uncased wells filled with rock or sand, operating on the same principle as the "French drain". They may be located beneath a backfilled trench or a shallow bed of filter material. Periodic removal and replacement of the trench and drain filter layers is usually necessary, as they eventually become sealed with water-borne silt and sediment. If trenches or sand drains extend below the highest level of a fluctuating ground water table, and are located in a silty soil, they should be completely backfilled with filter material to avoid silt migration from the adjacent soil, and subsequent clogging.

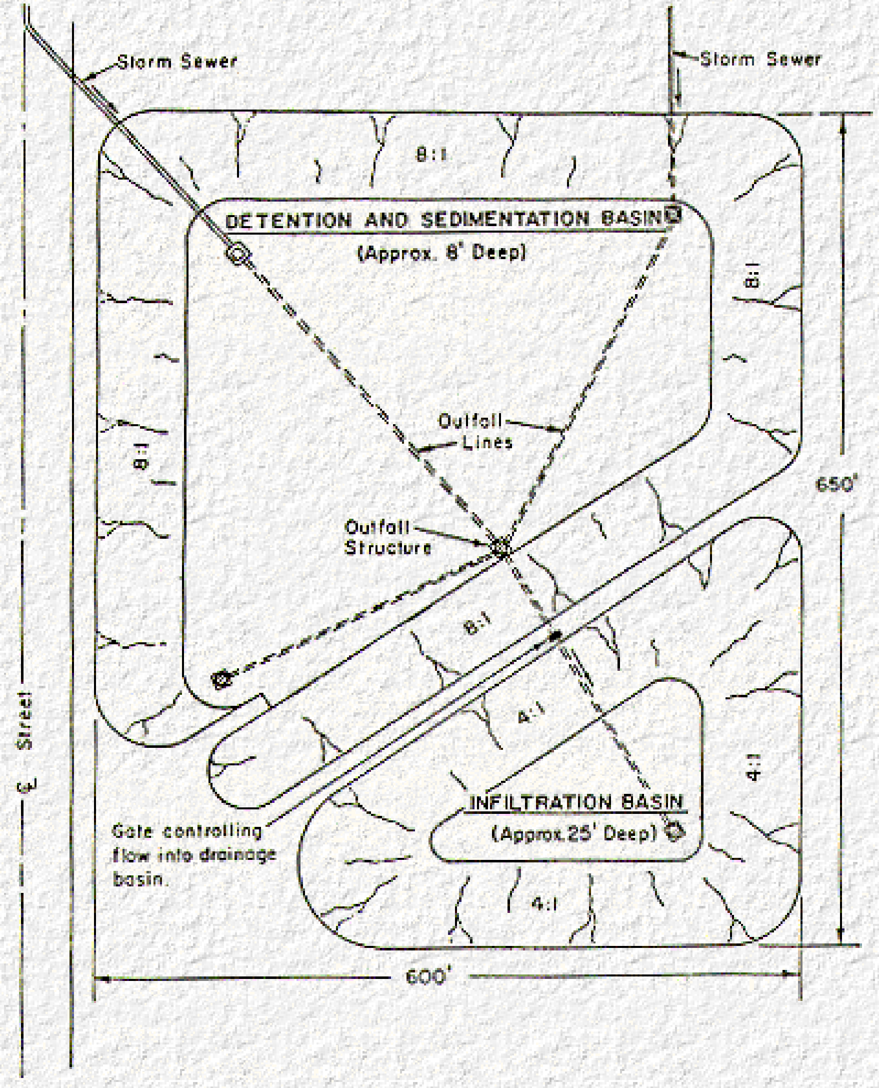

Figure 4-C-3. Typical Park-Type Infiltration Installation. ***(Fresno Metropolitan Flood Control District)***

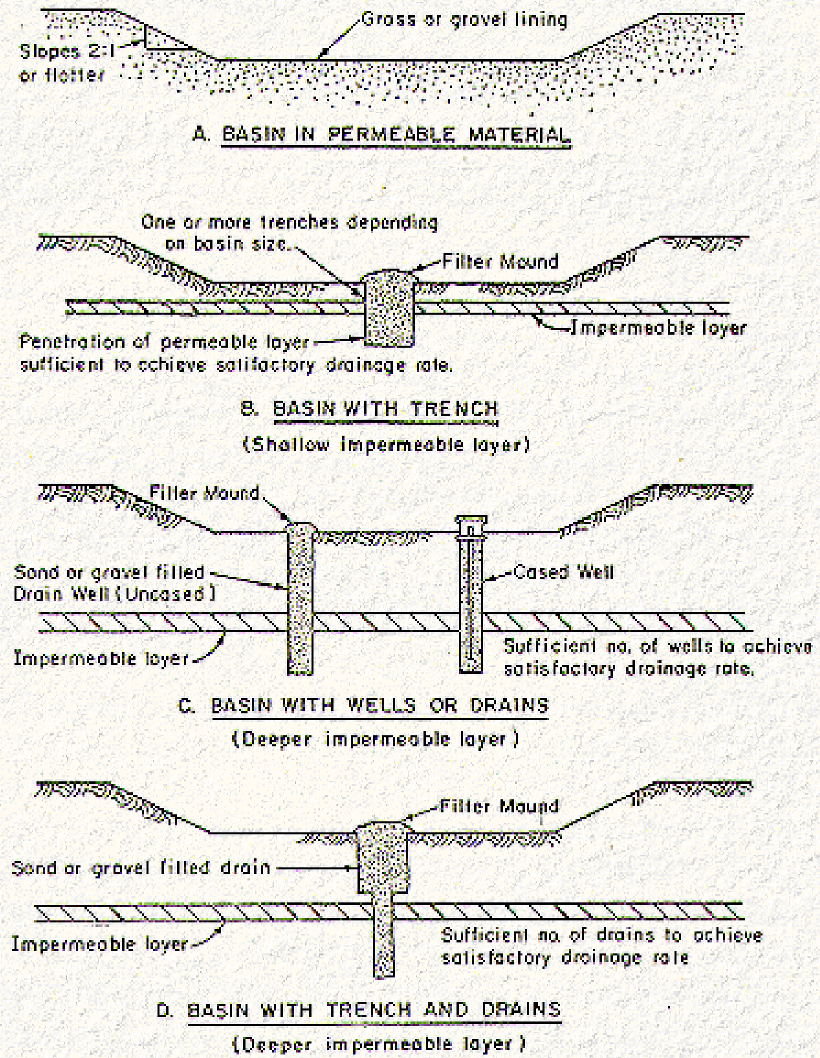

Figure 4-C-4. Typical Infiltration Basin Cross-sections. ***(Courtesy of Caltrans)***

Infiltration basins may be lined with filter material to help prevent the build-up of impervious silt deposits on the soil surface. A 6-inch (0.15 m) layer of pea gravel on the basin floor can serve to effectively screen out suspended solids and keep infiltration rates high. The gravel layer can be economically replaced or cleaned when it becomes clogged. However, planting of grass on basin sideslopes will extend infiltration efficiency by keeping soil pervious, and will reduce maintenance due to clogging and prevent erosion.

Grass serves as a good filter material, particularly the Bermuda variety, which is extremely hardy and can withstand several days of submergence. If silty water is allowed to trickle through Bermuda, most of the suspended material is removed within a few yards of surface travel. Well-established Bermuda on a basin floor will grow up through silt deposits, forming a porous turf and preventing the formation of an impermeable layer. Bermuda grass infiltration works well with long narrow shoulder-type basins (scales, ditches' etc.) where highway runoff flows down a grassy slope between the roadway and the basin. Bermuda requires little attention besides summer irrigation. Consideration could also be given to discing or spading a 6-inch (0.15 m) layer of coarse organic trash (such as cotton boll hulls, leaves, stems, etc.) into the basin floor to increase permeability. Another procedure, applicable to clay and hardpan surfaces, is the tilling, ripping, or scarifying of the top 2 to 3 ft (0.6 to 0.9 m) of soil to loosen the impervious upper stratum.

An example of infiltration basin specifications is provided In Table 4-C-1 to assist in designing basin facilities.

Table 4-C-1. Specifications for Basin Design (Fresno Metropolitan Flood Control District, California)

	Residential Area		**Industrial Area**
	Recreation Use	**Recharge Use**	
Depth	10-15 ft	10-20 ft.	15-25 ft.
Cut Slopes	6:1 to 8:1	6:1 to 8:1	4:1 to 6:1
Landscaping	Bermuda mix on sides and bottom, some trees on sides and top	Bermuda mix on sides only some trees on sides and top	Barbuda mix on sides only; trees optional
Design volume for a 100-year, 10-day storm equals 6.6 inches.			
Volume may be reduced according to quantity expected to be recharged over the 10-day design period. For the design volume, the Rational Method is used, where Volume = CiAt			
C = i = A = t =	Composite runoff coefficient Average rainfall intensity over the design period, in inches per hour Drainage area, in acres Design period of rainfall, In hours		
Note: I inch = 25.4 mm 1 ft. = 0.305 m			

b. Operating Head

As infiltration is directly proportional to basin head under all conditions, best results are achieved by utilizing the highest values of operating head that are consistent with other design requirements, such as side slope stability and access for surface maintenance.

Site topography, inlet location, and gradient are used to establish the basin bottom elevation and permissible water depth. The average operating head for the design storm can be assumed as the maximum water depth divided by 2. This value is used to determine the unit area infiltration/time curve for the basin.

Various procedures are available for design of basins of proper capacity. Where soil infiltration rates are low, basins are designed as retention systems and any available infiltration of storm water from basin into soil is an added factor of safety.

c. San Joaquin County, California Procedure

Volume of storage is computed from the basic formula:

V = CAR/12

Where V is the required volume of basin in acre feet
C is the runoff coefficient as defined in Chapter 4-B
A is the contributing drainage area in acres
R is the total rainfall (I x t) in inches for the design storm period, where

I = Intensity in inches/hour, and t = length of storm in hours. In urbanized areas R is equivalent to a 10 year, 48 hour storm. In rural areas R is equivalent to a 10 year, 24 hour storm.

These basins can be designed with outlet facilities providing terminal drainage capable of emptying a full basin within 24 hours in urban areas and within 48 hours in rural areas.

---

d. New York State DOT Procedure

This procedure sizes the infiltration basin as a temporary reservoir to hold only the peak volume of water for the period of design storm inflow. The inflow is a mass inflow curve. The mass inflow curve was developed in Example Problem 1, taken from the New York DOT Research Report 69-2(2). The steps are:

- Step 1: Determine drainage area and proper coefficient of runoff.
- Step 2: Select appropriate rainfall intensity duration-frequency curve and construct a mass inflow curve of rainfall for design of the basin.
- Step 3: Adjust mass inflow curve using coefficient of runoff selected in Step 1 to reflect mass inflow curve for runoff.
- Step 4: Final adjustment of mass inflow curve as per Example Problem 1.

The optimum basin size is selected to provide a temporary reservoir and operate at the desired head.

EXAMPLE PROBLEM 1

FIND: The mass inflow curve of runoff to the recharge basin for the 50-yr design storm.

- STEP 1: From the storm drain calculations (Table A), the following data are obtained:
  . Drainage area = 12.57 x 43,560 = 548,000 sq ft
  b. Weighted C_{avg} factor = ΣCA/A = 8.26/12.57 = 0.66.

- STEPS 2 and 3: From the National Weather Service curves for New York City (Figure B), mass inflow curves of rainfall and runoff are computed (Table B) and plotted (Figure C).

- Step 4 A vertical dimension equal to 46,000 cu ft (A/12) is measured downward at several points along the mass curve of rainfall, and a new curve plotted as indicated by the dashed line in Figure C. This revision of the mass inflow curve is made to assure that total losses in the drainage basin do not exceed 1 in. of rainfall. In this example, the revised curve is above the mass curve of run off. The design mass inflow curve for the basin (Figure D) should therefore follow the plot of the revised curve of Step 4. If the curve developed in Step 4 fell below the mass curve of runoff, no revision should be made. The design mass inflow curve for this condition would be equal to the mass curve of runoff in Step 3.

EXAMPLE PROBLEM 1: MASS INFLOW CURVE DETERMINATION(2)

GIVEN: A storm drain system (Figure A) located near New York City, and the following storm drain design calculations:

Table A. Storm Drain Design Calculations

Station		Drainage Area, acres				Time			Rainfall, in./hr.	Discharge, cfs	Culvert			Velocity, fps	Q Flowing Full
											Length, ft	Slope, (1:1)	Size, in.		
Catch Basin	Pipe Run	Sub-area	C	SCA	Acc. SCA*	Min. t_c	Min. t_p	t_c+t_p	i_{10}	$Q_{10}=CAi$	L	S	D	V	
5	--	6.67	0.6	4.00	4.00	20	0.0	20.0	4.1	--	--	--	--	--	--
	5-4	--	--	--		--	0.2[a]	--	--	16.4	100	0.0100	24	7.0	7.0
4	--	2.00	0.70	1.40	5.40	--	--	20.2	4.1	--	--	--	--	--	--
	4-3	--	--	--	--	--	0.3	--	--	22.1	180	0.0081	30	7.0	40.0
3	--	1.07	0.70	0.75	6.15	--	--	20.5	4.1	--	--	--	--	--	--
	3-1	--	--	--	--	--	0.8	--	--	25.2	300	0.0064	30	6.3	35.0
2	--	2.00	0.75	1.50	1.50	15	0.0	15.0	4.8	--	--	--	--	--	--
	2-1	--	--	--	--	--	0.8	--	--	7.2	200	0.0049	24[b]	4.4	14.0
1	--	--	--	--	--	--	--	15.8	--	--	--	--	--	--	--
1	--	0.83	0.74	0.61	8.26	--	--	21.3	4.0	--	--	--	--	--	--
	1-RC	--	--	--	--	--	0.7	--	--	33.1	300	0.0064	30	6.8	35.5
Recharge Basin		**12.57 total**			**8.26**			**22.0**							

* Accumulated SCA.
[a]Sample calculation: t_p = 100/(7x60) = 0.2
[b]Minimum size for maintenance purposes.

Comment: Comment: Table B. Calculations for Step 2 (Col. 4) and Step 3 (Col. 5)

Table B. Calculations for Step 2 (Col. 4) and Step 3 (Col. 5)

	Rainfall Intensity,	Depth on Area,	Mass Inflow Curves	
			Rainfall,	Runoff
Time	in./hr[a]	ft	cu ft[c]	cu ft[d]
1	2	3	4	5
5 min	8.5	0.059[b]	32,400	21,400
10 min	7.2	0.100	54,800	36,200
20 min	5.5	0.153	83,800	55,300
30 min	4.5	0.188	102,800	67,800
60 min	3.0	0.250	137,000	90,400
2 hr	1.9	0.317	173,500	114,400
3 hr	1.40	0.350	191,800	126,500
4 hr	1.13	0.377	207,000	136,500
5 hr	0.95	0.396	217,000	143,200

[a]New York City 50-yr frequency.
[b]Sample calculation: 8.5 x 5/60 x 1/12 = 0.059.
[c]Col. 3 x drainage area, sq. ft.
[d]Col. 4 x C_{avg}.

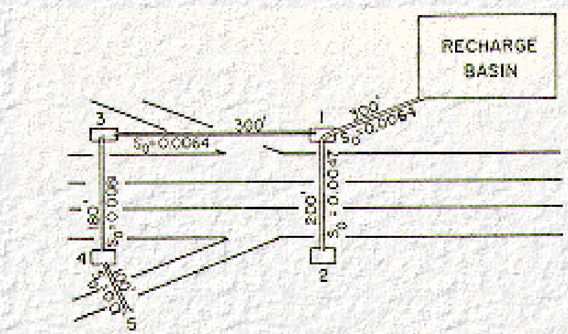

Figure A. Storm drain system layout, including five catch basins and five pipe runs

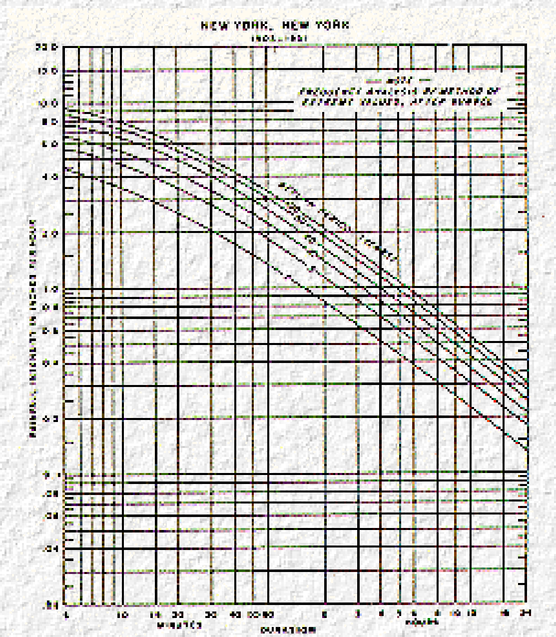

Figure B. New York City rain intensity-duration-frequency curves, 1903-1951, as compiled by the National Weather Service

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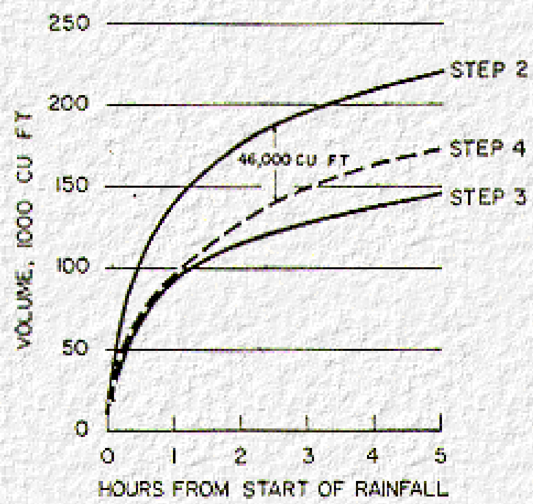

Figure C. Construction of mass

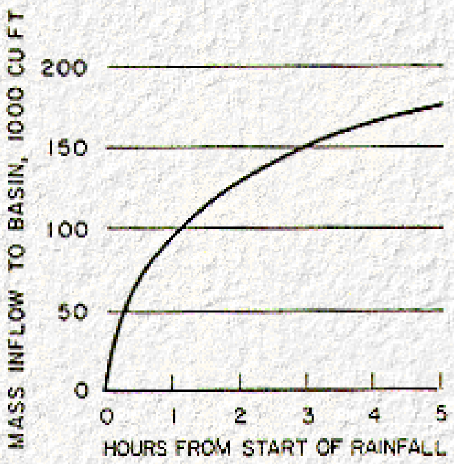

Figure D. Design mass inflow curve inflow curves. for 50-year frequency of rainfall.

Example Problem 2 which is also taken directly from New York DOT, Research Report 69-2 serves to illustrate the use of the mass inflow curve for basin size selection. The method is essentially one of graphical rate comparison leading to values of plane flow area and volumetric capacity required for the basin to operate at peak head H. This procedure contains a net safety factor of approximately 1.4 and basins designed by this procedure will be 73-percent full for the design storm, and possess a 27 percent reserve volume for contingencies.

EXAMPLE PROBLEM 2: BASIN SIZE DESIGN(2)

GIVEN: Coordinates of the mass reflow curve for a proposed basin:

Q_i, million cu ft			Q_i, million cu ft		
Hours	Actual	Corrected	Hours	Actual	Corrected
1	2	3	1	2	3
1	0.08	1.00	10	6.48	8.00

2	0.16	2.05	11	7.10	8.19
3	0.50	3.00	12	7.60	8.35
4	1.11	408	13	7.94	8.41
5	1.82	5.02	14	8.16	8.42
6	2.80	5.85	15	8.27	8.42
7	3.79	6.85	16	8.37	8.42
8	4.80	7.22	17	8.42	8.42
9	5.74	7.68	18	8.42	8.42

FIND: The size and dimensions of a square basin plan for an H'=20 ft, using an infiltration equation for the site of

$$Q = 12.02\sqrt{t} \bullet A$$

NOTE: The equation represents infiltration under an average head of 10 ft, since the maximum operating head H' is given as 20 ft.

STEP 1: Actual mass inflow curves (such as determined from field instrumentation) usually will have a tail or inflection at early values of time. The tabulated data have this characteristic, which requires a correction to time coordinates-in effect removing this tail for proper comparison with mass infiltration curves. The method of establishing this time correction is shown in Figure A. For tabulated data, the resulting time correction is 3.3 hr. If the mass inflow curve is calculated from the procedures in Example Problem 1, no tail is obtained and no time correction is necessary,

STEP 2: Replot or trace the corrected mass inflow curve on a new sheet of graph paper (Figure B) to permit convenient superposition of mass infiltration curves for various assumed areas.

STEP 3: Compute the infiltration per unit area (e.g., 1, 100, 1,000 etc.) for convenient values of time from the infiltration equation given for the site throughout the time period of interest.

STEP 4: Assume at least four values of the flow area for which Q will bracket within the range of 0.50Qi to 0.85Qi at the time when the corrected mass-inflow curve starts peaking. Multiply the unit area infiltration Q coordinates (from Step 3) by the appropriate factor to obtain a "mass infiltration" curve for each value of assumed flow area. Plot these curves on the graph prepared in Step 2, using the same coordinates. Figure B shows the final plot as it will appear at the end of this step.

STEP 5: In Figure B. scale off the peak differential between the mass inflow curve and each of the infiltration curves. This is most conveniently done with dividers. Tabulate the information as follows:

Area, A_f	Peak ΔQ	H'=Q/A_f
100,000	4.25 x 106	42.5
112,000	3.70 x 106	33.0
125,000	3.25 x 106	26.0
150,000	2.35 x 106	15.7

STEP 6: Graphically plot these At values against H' and draw the curve as in Figure C. Locate the curve intersection with the desired value of H'. The abscissa coordinate of this intersection is the plane flow area (Af) required for the basin, in this case 138,000 sq ft.

STEP 7: Multiply the plane flow area obtained in Step 6 by the design value of H' to obtain the design value for basin volume, in this case 2.76 x 106 cu ft.

STEP 8: Assume convenient rounded values for the area of the water surface As' in the approximate interval 1.1 Af <As <1.3 Af. For these values of As, calculate the basin volume for depth H' (20 ft in this example) and the desired side slope. The following table provides simplified formulae for square, rectangular, or circular basin plans.

<table>
<tr><th>Side Slope (h:v)</th><th>P_b</th><th>A_s-A_b</th><th>Basin Volume</th></tr>
<tr><td>3:1</td><td>P_b-24H'</td><td>3H'(P_b+12H')</td><td rowspan="4">$$\frac{H'}{3}\left(A_b + A_s + \sqrt{A_b A_s}\right)$$</td></tr>
<tr><td>2:1</td><td>P_b-16H'</td><td>2H'(P_b+8H')</td></tr>
<tr><td>1.5:1</td><td>P_b-12H'</td><td>1.5H'(P_b+6H')</td></tr>
<tr><td>1:1</td><td>P_b-8H'</td><td>H'(P_b+4H')</td></tr>
<tr><td>Where:</td><td colspan="3">P_b = perimeter of basin bottom, P_s = perimeter of water surface at depth H', A_b = area of basin bottom, and As 5 area of water surface at depth H'.</td></tr>
</table>

Assuming side slopes of 2:1 for the example problem, the relationship between As and basin volume is readily computed to be as shown in Figure D. Using the design volume obtained in Step 7, obtain A_s = 168,500 sq ft.

STEP 9: Design summary. The basin size and dimensions are as follows:

Top:	411 ft x 411 ft. Area = 168,500 sq ft
Floor:	331 ft x 331 ft. Area = 109,140 sq ft
Side Slopes:	2 horizontal to 1 vertical
Volume:	2,760,000 cu ft
Depth:	20 ft.

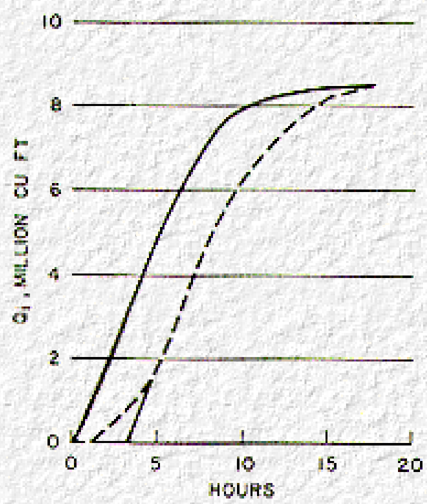

Figure A. Time correction for mass in- flow curves with tail; corrected curve constructed by shifting all time coordinates 3.3 hr to left

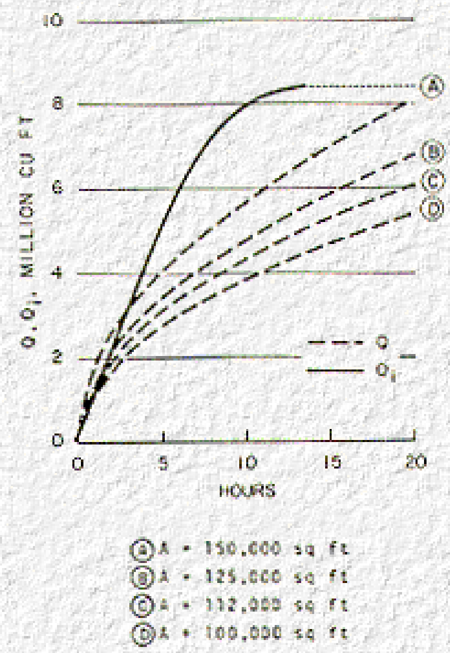

Figure B. Comparison of inflow-outflow rates

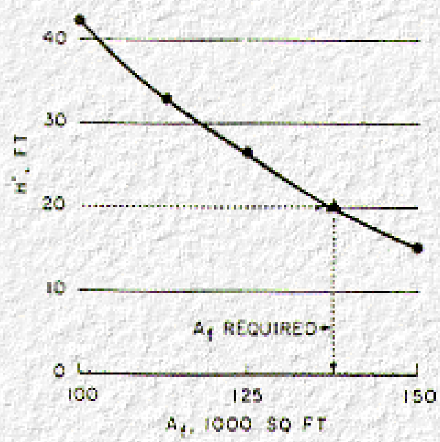

Figure C. Determination of required flow areas, A_f

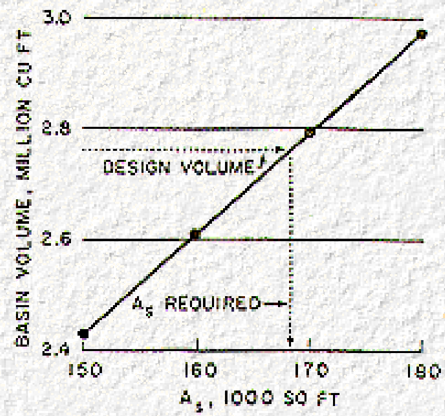

Figure D. Final basin sizing.

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3. Trenches

a. Design Considerations

In conventional or positive outfall drainage system design, the objective is to determine peak flow quantities and velocities for proper sizing of various elements in the system. For infiltration trench design, the inflow into the trench with time is the object of interest. Hydraulic design for infiltration trenches requires determining the time-related inflow and infiltration rates.

The design of the system should include consideration of both storage and soil infiltration inherent in the system. The infiltration capacity required for the storm water disposal will usually be considerably less than the peak inflow rate because of storage in the system.

The first step in developing the time-related inflow rate is to use appropriate rainfall intensity-duration-frequency curves or other procedures that are representative of the particular area of study, as discussed in Chapter 4-B.

The selection of the appropriate storm frequency is governed by the type of facility and criteria established by the controlling agency.

---

b. Infiltration Rate

Using the results of percolation or infiltration testing (as determined by methods described in Chapter 4-A) soil infiltration rates can be calculated for a reasonable hydraulic design head. The infiltration rate can be equated to cubic feet per second per linear foot of trench for sizing the particular facility. Specific rates of water exfiltrated from the trench and infiltrated into the soil are determined for each increment of depth.

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c. Trench Storage

For trench design, the volume of available storage area can be calculated based on usable void area above the water table. This storage volume is represented as the vertical distance between the mass inflow curve and the storage volume curve as defined below in Example 3.

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d. Example Problems

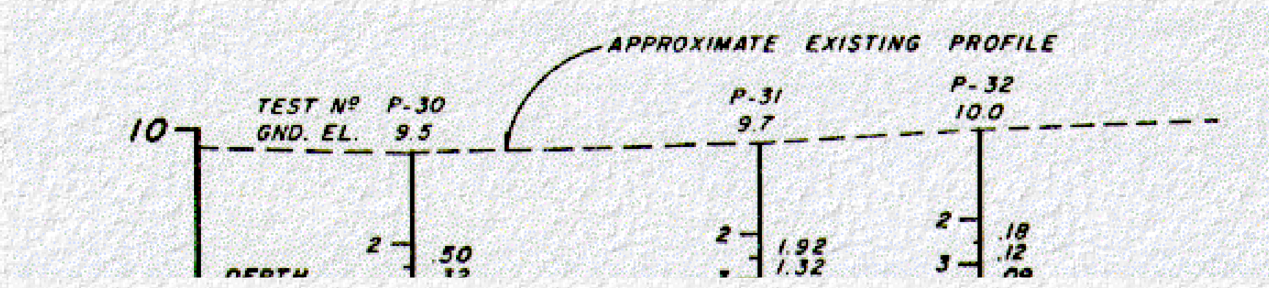
Example 3

The problem involves the disposal of storm water pending along the alignment of a main thoroughfare in Miami, Florida. Existing soil conditions are generally acceptable for an infiltration drainage system. Subsurface exploration indicates sandy soils. Space limitations suggest the use of an underground trench system.

Infiltration results from tests made along a portion of the proposed trench alignment are shown in Figure 4-C-5.

The drainage area is comprised of 1.83 acres of pavement and 1.28 acres of grass. Indicated coefficients of runoff are: C_1 = 0.9 for pavement and C_2 = 0.2 for grass.

The trench is to be sized using the mass inflow curve and the infiltration rate for cumulative exfiltration from the trench. Assume a 3-year design storm frequency, for Miami Florida. The hourly rainfall intensities are selected from Figure 4-C-6 and utilized to determine the elements of the mass inflow curve. The resulting mass inflow curve and cumulative exfiltration line are shown in Figure 4-C-7 for the drainage area between Stations 180+00 and 189+00.



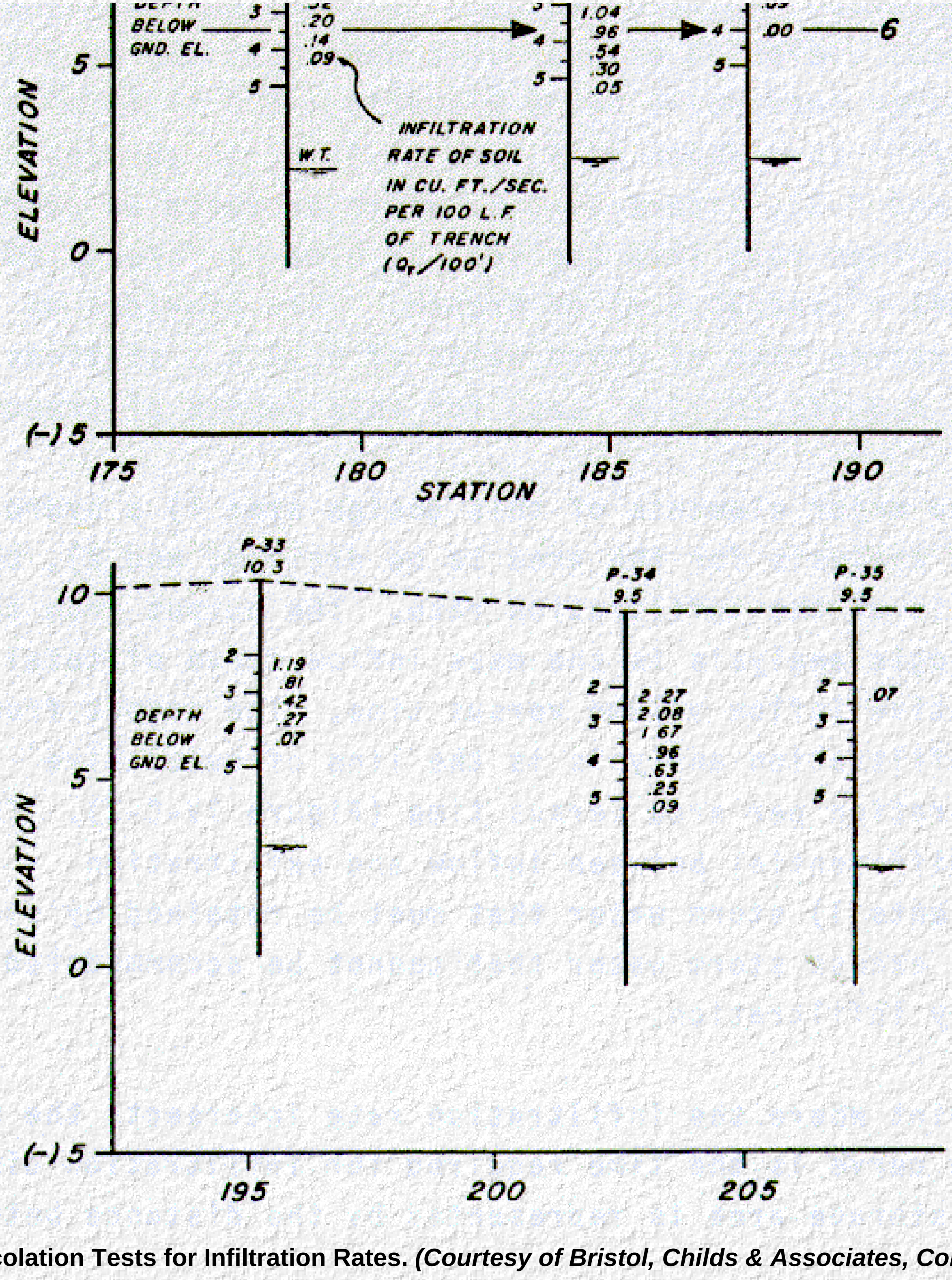


Figure 4-C-5. Percolation Tests for Infiltration Rates. ***(Courtesy of Bristol, Childs & Associates, Coral Gables, Florida)***

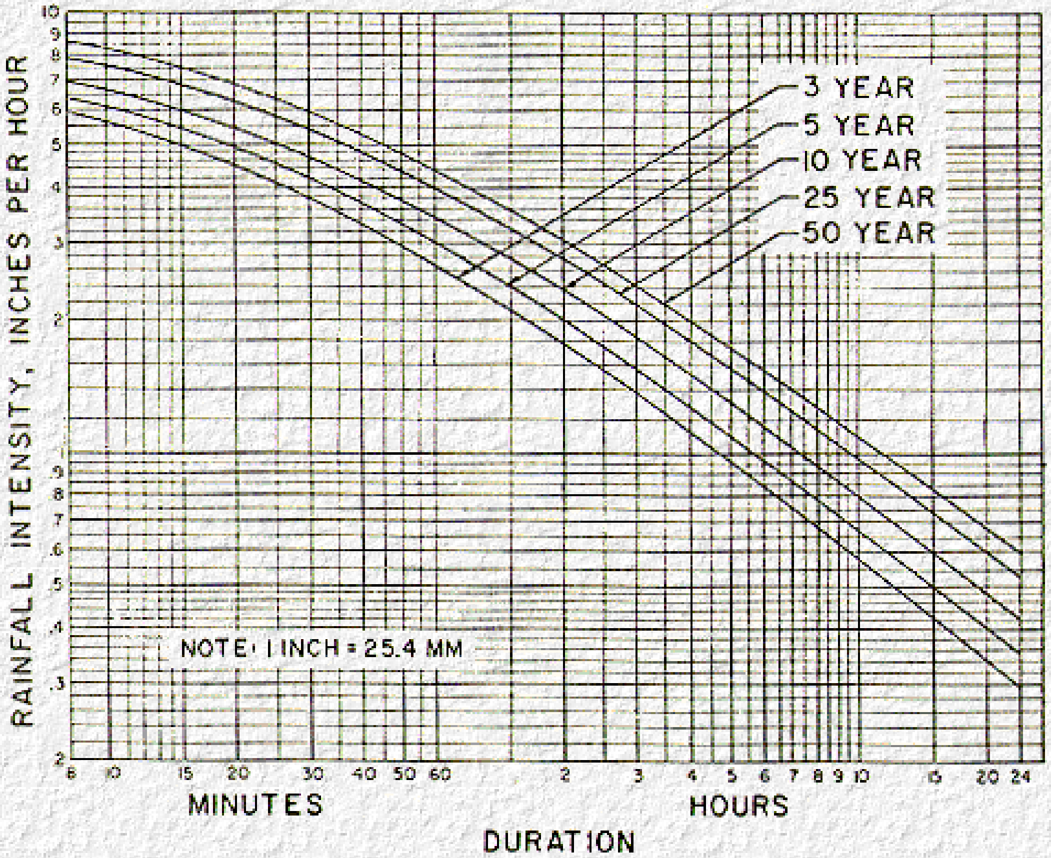


Figure 4-C-6. Rainfall Intensity-Duration-Frequency Curves for Zone 5, Miami. ***(From Florida DOT)***

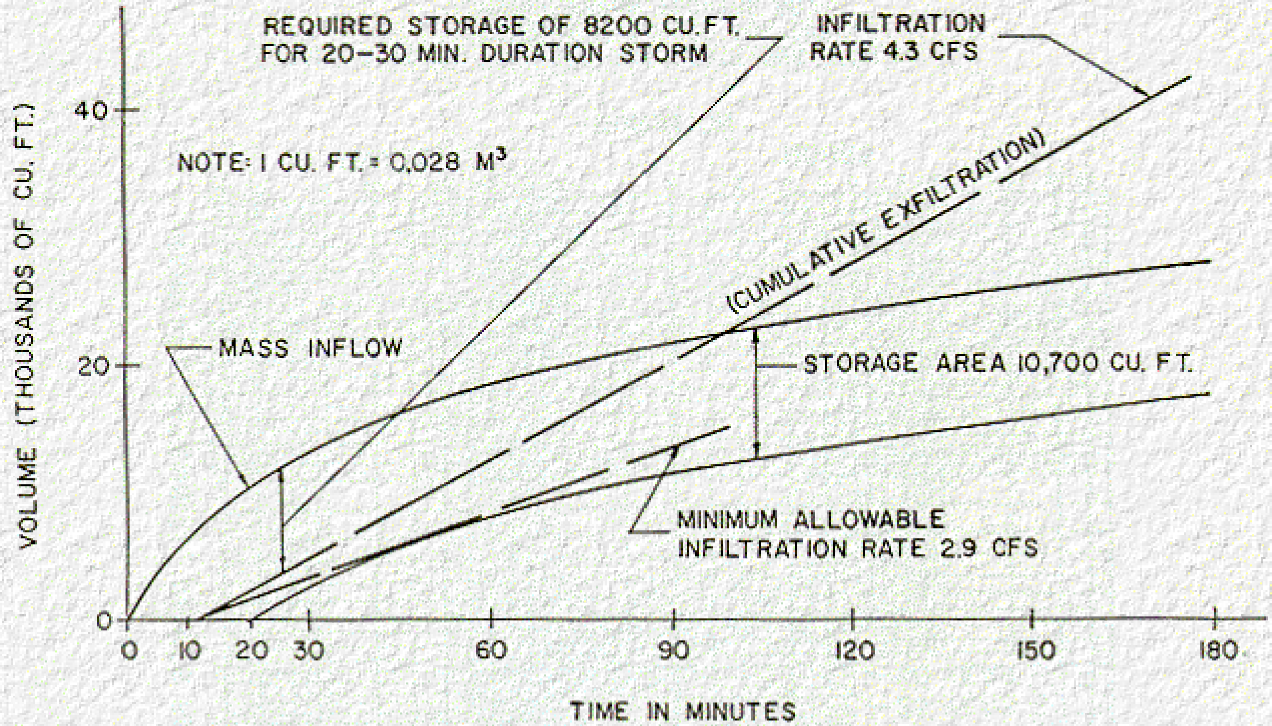


Figure 4-C-7. Mass Inflow and Cumulative Exfiltration Curve/ *(Courtesy of Bristol, Childs & Associates, Coral Gables, Florida)*

The estimated infiltration rate is based on the highest practical elevation of hydraulic head that can be obtained. The design head would be at Elevation +6.0 (1.83 m) or equivalent to a depth of 4 feet (1.22 m) in the test hole (P-31 and P-32, Figure 4-C-5) which produced an average infiltration rate into the soil of 0.48 cu ft/sec/100 L.F. (0.01344 m^3/sec/30.6 m) of trench. This provides an exfiltration rate of storm water of 0.48 x 9 stations which equals 4.3 cfs (0.12 m^3 sec) for this length of trench.

The two major elements of this design are: 1) a hydrologic runoff analysis for the area to be drained; and 2), the analysis of the infiltration rate. The output from the hydrologic analysis is the mass inflow curve of total cumulative inflow volume versus time. The output from the infiltration analysis is the line of cumulative exfiltration per area versus time (Figure 4-C-7). The peak differential between inflow and exfiltration represents 1) storm water that must be retained by storage; or 2), excess storm water that cannot be accommodated into soil by infiltration.

The point where the infiltration rate intersects the mass inflow curve is the time required for infiltration disposal. Total storage area is represented by the distance between the total mass inflow curve and a curve concentric to the mass inflow curve. The peak differential between these curves represents the total volume of water, greater or less, than that which is infiltrated into the soil.

The proposed trench cross-sectional area is shown in Figure 4-C-8. The total available storage area in the trench is calculated based on void space above the design water table. For this particular problem, the finish grade elevation is +9.0 (+2.75 m), and the design water table is located at Elevation +2.5 (+0.76 m) as shown in Figure 4-C-8. The available storage area above the water table is calculated as follows:

**From Table 84, Handbook of Hydraulics, H. W. King, 4th Edition, McGraw-Hill. (For circular conduit flowing part full.)*

Storage = 2.50 + 3.53 + 3.97 + [3.53 -0.08861*(3)^2]-[(0.5) (5) -0.08861*(3)^2] 0.5 = 11.89 cu ft/L.F. (1.09 m^3/sec/m).

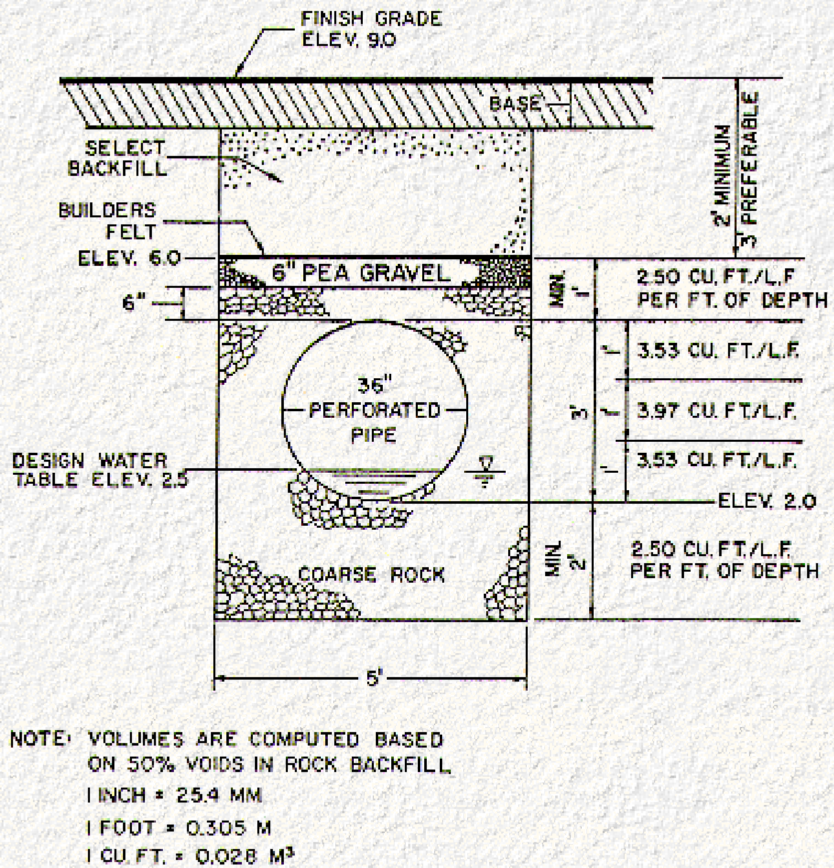


Figure 4-C-8 Detail Sowing Volume of Storage in Infiltration Trench. ***(Courtesy of Bristol, Childs & Associates, Coral Gables, Florida)***

Length of trench between Stations 180+00 and 189+00 = 900 ft (274.5 m). Total available storage = 11.89 (900) 10,701 cu ft. Say 10,700 cu ft (299.6 m^3).

The most critical time is between 20 and 30 minutes duration, when the maximum storage area of 8,200 cu ft (230 m^3) is required, as shown in Figure 4-C-7. The design safety factor,

$$SF = \frac{\text{Available Infiltration Rate}}{\text{Minimum Allowable Infiltration Rate}} = \frac{4.3 \text{ cfs}}{2.9 \text{ cfs}} = 1.48$$

The trench design is thus considered satisfactory.

Example 4

Using the data from Example 3, determine the trench length necessary to exfiltrate the storm inflow assuming no storage is available in trench. The time duration of storm is assumed as 20 minutes.

From Example 3:

$A_1 = 1.83$, $C_1 = 0.9$

$A_2 = 1.28$, $C_2 = 0.2$

$CA = (0.9)(1.83) + (0.2)(1.28) = 1.90$

From Figure 4-C-6, i = 4.4 inches/hour (112 mm/hr) for 3-years frequency storm at t = 20 minutes.

$Q = CAi = 1.90 (4.4) = 8.4$ cfs (0.24 m^3/sec).

The maximum average exfiltration rate is 0.0048 cfs/L.F. (0.00044 m^3/sec/m) of trench.

Required trench length, $L = \frac{8.4 \text{ cfs}}{0.0048} = 1750$ ft (534 m).

The trench length requirement for this problem is double that of Example 3 when storage is neglected.

---

e. Specification Guidelines for Infiltration Trenches

---

(1) General

The trench cross-section shown in Figure 4-C-8 for Example 3 is typical of most installations in southern Florida and is applicable in other areas where soil provides sufficient infiltration capacity. Even where infiltration rates are marginal, the system could supplement the drainage requirements of a positive outfall system by storing, and infiltrating a portion of the storm water into the soil; thereby reducing the downstream requirements of the positive system.

---

(2) Perforated or Slotted Pipe

Pipes manufactured of steel, aluminum, concrete, or other materials, are available for this application. Perforated metal pipes are required to have 3/8 inch (9.5 mm) diameter perforations uniformly spaced around the full periphery of a pipe. Specifications stipulate not less than 30 perforations per square foot (0.305 m^2) of pipe surface. Other perforations not less than 5/16 inch (8.0 mm) in diameter, or slots, are permitted if they provide a total opening area of not less than 3.31 square inches (2135 mm^2) per square foot (0.305 m^2) of pipe surface.

Tentative specifications for slotted concrete pipe with cast slots have been developed based on field performance and cooperative testing by Florida DOT and industry. Concrete pipe with 3/8 inch (9.5 mm) wide slots have been specified as illustrated in Figure 2-9 (Chapter 2). Tentative specifications stipulate that cast slots shall be circumferential in direction, not less than 3/8 inch (9.5 mm) in width and not less than 4 inches (101.6 mm) in length at the inside of the pipe. Four rows of slots are specified for pipe 30 inches (0.76 m) in diameter and less and six rows are specified for pipe 36 inches (0.92 m) in diameter and larger.

The liberal number of holes insure free and rapid flow in and out of the walls of the pipe. Large-sized pipe adds to total storage in the trench. The use of a pipe in the trench system also allows for ease of maintenance as described in Chapter 6. The pipe serves as a catchment for sediment without reducing overall efficiency.

---

(3) Pipe Backfill

Backfill material for infiltration trenches should meet the aggregate gradation for one of the ASTM size numbers presented in Table

4-C-2. These gradations provide satisfactory performance in underground storm water disposal systems. These aggregates serve to reduce exit velocity through pipe perforations or slots and provide sufficient void space for storage and movement of water. Aggregates for this purpose must be sound and comply with an established specification for durability.

The above aggregates provide sufficient void space to allow the normally encountered fine sands, silts, silty clays, and other fine material in storm water to pass through the perforations or slots in pipe conduit into the backfill. When fine native materials are encountered in the excavation, consideration should be given to placement of a filter cloth envelope around the backfill to prevent migration of these fine materials that could result in possible contamination and clogging of the backfill during reverse flow conditions resulting from high groundwater. A number of plastic woven or non-woven filter fabrics can be used for this purpose at relatively moderate cost. Refer to Section 6, "Filter Cloth" of this Chapter and also Appendix E-2 for further information on filter fabrics. For construction details refer to Chapter 5.

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(4) Pea Rock or Gravel

This material is placed in a 6-inch (0.15 m) layer over the top of the aggregate for the pipe backfill. This layer serves as a filter below the impervious barrier (Builders Felt). The aggregate gradation for this layer shall consist of material passing the 1-inch (25 mm) sieve with not more than 5 percent passing the No. 4 (4.75 mm) sieve.

Table 4-C-2: ASTM Standard Sized of Coarse Aggregate for Underground Disposal of Water Designation: D448-54 (Reapproved 1973) Amount Passing (Square Openings), weight percent

Size Number	Nominal Sizes	4-in. (100-mm)	3 1/2-in. (90-mm)	3-in. (75-mm)	2 1/2-in. (63-mm)	2-in. (50-mm)	1 1/2-in (37.5-mm)	1-in (25.0-mm)	3/4-in. (19.0-mm)	1/2-in. (12.5-mm)	3/8-inch (9.53-mm)	No. 4 (4.75-mm)
1	3 1/2 to 1 1/2-inch (90 to 3.75-mm)	100	90 to 100		25 to 60		0 to 15		0 to 5			
2	2 1/2 to 1 1/2-inch (63 to 37.5-mm)			100	90 to 100	35 to 70	0 to 15		0 to 5			
24	2 1/2 to 3/4-inch (63 to 19.0-mm)			100	90 to 100		25 to 60		0 to 10	0 to 5		
3	2 to 1-inch (50 to 25.0-mm)				100	90 to 100	35 to 70	0 to 15		0 to 5		
357	2-inch to No. 4 (50 to 4.75-mm)				100	95 to 100		35 to 70		10 to 30		0 to 5
4	1 1/2 to 3/4-inch (37.5 to 19.0-mm)					100	90 to 100	20 to 55	0 to 15		0 to 5	
467	1 1/2-inch to No. 4 (37.5 to 4.75-mm)					100	95 to 100		35 to 70		10 to 30	0 to 5

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4. Wells

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a. Design Considerations

As discussed in various chapters of this manual, many terms are used to describe infiltration systems utilizing wells; i.e., drain well, dry well, rock well, recharge well, gravity well, and infiltration well. These types are similar, with certain minor modifications in each case. The most commonly used is the gravity well. Other types of wells are also discussed in Chapter 2, "State-of-the Art". However, they are not considered applicable to most drainage situations encountered.

Wells require little space, and may be designed with a very simple surface structure or drop inlet. They clog very easily, however, when storm water contains silt or sediment; and cleaning, or restoration can be difficult. For maintenance considerations refer to Chapter 6.

Cased wells are the most effective when water is of good quality, and if the installations are properly maintained. The casing usually consists of perforated or slotted pipe running the depth of the well shaft. It is recommended that perforations and slots be limited to permeable soil strata, in order to minimize the amount of silt and fine soil washing into the well through these openings. It is desirable to place several inches (0.15 to 0.25 m) of gravel packing around drain well casings to act as a conveyor of water to the well side and as a filler between the pipe and the well wall. Coarse sand or aggregate packing is easier to clean when the compressed-air jet redevelopment method is used.

When selecting pervious backfill for drain wells, it is important to realize that certain gradings that fall within the typical specification limits for ordinary permeable materials may be relatively impervious with respect to this application. Also, some materials which are ordinarily considered free draining, are in fact, guise impermeable. Some beach and concrete sands fall in this classification. This factor is often overlooked and drains may be filled with materials which actually inhibit drainage. Ordinarily, the added expense required to provide a properly graded aggregate which would insure good permeability is negligible in the case of a drain well. Acceptable aggregate gradations are shown in Table 4-C-2.

Gravity wells are generally augured wells. They are generally used for the infiltration drainage of small areas, or are used in conjunction with trench or basin systems to penetrate impervious layers that hinder percolation.

Typical well diameters usually vary from 2 ft (0.61 m) to 4 ft (1.22 m). Selection of size is based on local experience, soil conditions, and capability of available construction equipment.

To insure adequate drainage capacity, the well depth should reach pervious material. If the storm runoff water is to be properly treated, and with the approval of the local public health office, the aquifer should be partially penetrated to achieve the best drainage and facilitate maintenance. If no treatment is planned, the shaft must be terminated some distance above the water table. The local or state public health agency should be consulted regarding the amount of aquifer clearance required.

If a 10 ft (3.05 m) aquifer clearance is stipulated, as is often the case, the groundwater surface must be at least 20 ft (6.1 m) deep with a minimum well depth of 10 ft (3.05 m) for the well to perform satisfactorily, unless installed in a very permeable soil. If no groundwater is encountered, northern California experience indicates that 50 to 75 ft (15.3 to 22.9 m) is sufficient to reach sand deposits.

A sufficient number of wells should be drilled in a given area so that, all standing water is drained away within a reasonable amount of time. If wells are situated in shallow ditches or ponds within the median strip or immediately adjacent to the road shoulder, standing water is a hazard and should generally be completely removed in 24 to 48 hours following a storm. For wells being used in conjunction with drainage basins, the time limit is of less importance, but the combined infiltration rate should still average 0.5 ft per day (0.15 m/day) or greater. Typical installations in drainage basins are shown in Figure 4-C-4. Other drain well installations are shown in Figures 4-C-9 through 4-C-12.

Also available are various patented drywall or gravity well systems which utilize special hardware and precast concrete pipe with slots, or round openings and cast-in-place concrete boxes and catch basins.

Wells provide minimal storage and should be situated in a ditch, swale, or pond; or along curb and gutter, where overflow can be temporarily contained. Large diameter underground sumps or tanks located adjacent to smaller well shafts may provide another solution to temporary storage. Examples of detention and overflow storage are presented in Figures 4-C-2A and 4-C-2B of this chapter.

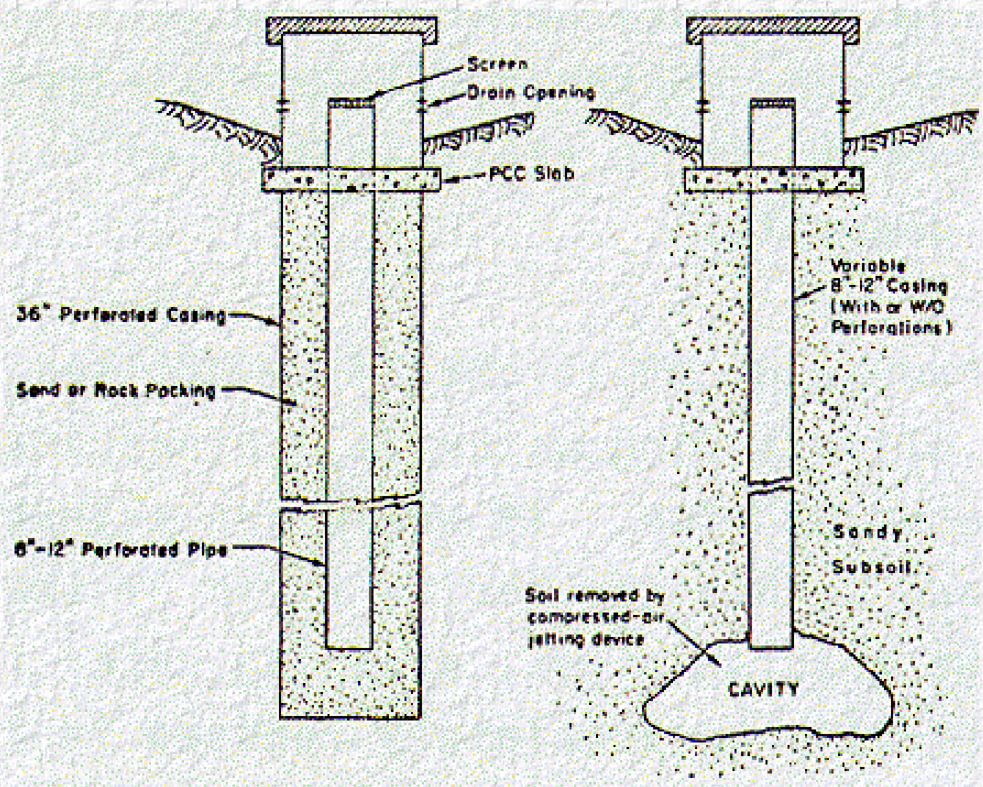


Figure 4-C-9. Drain Wells. ***(Typical California Installations)***

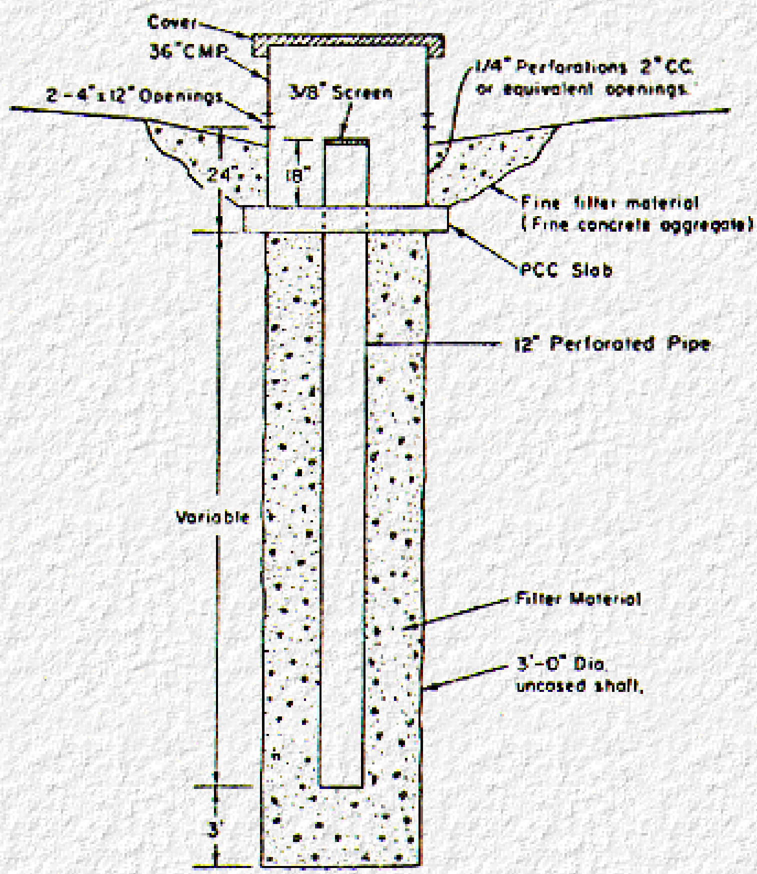


Figure 4-C-10. Drain Well*. (Typical California Installations)*

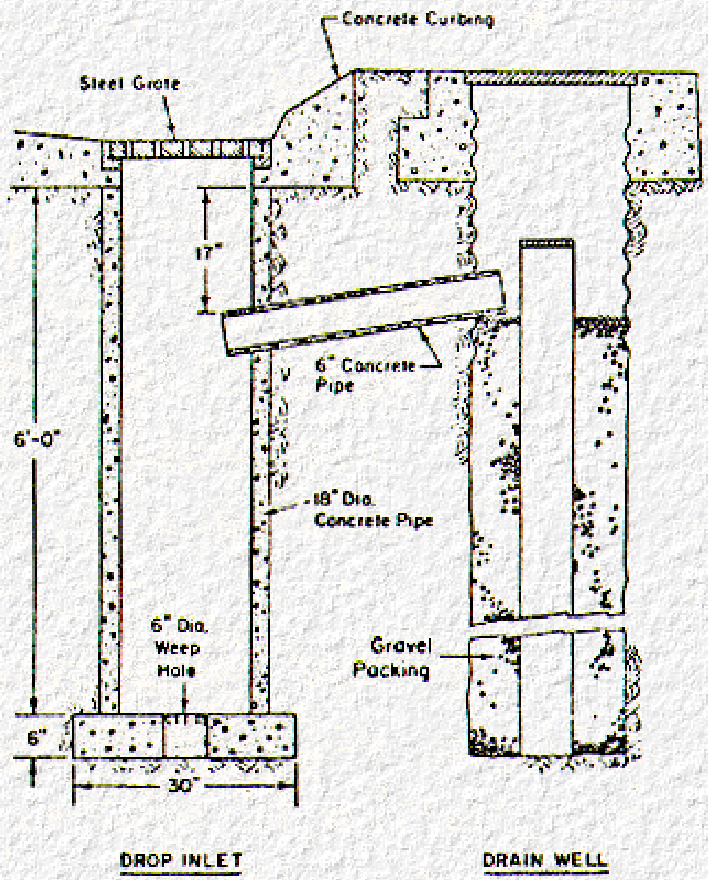
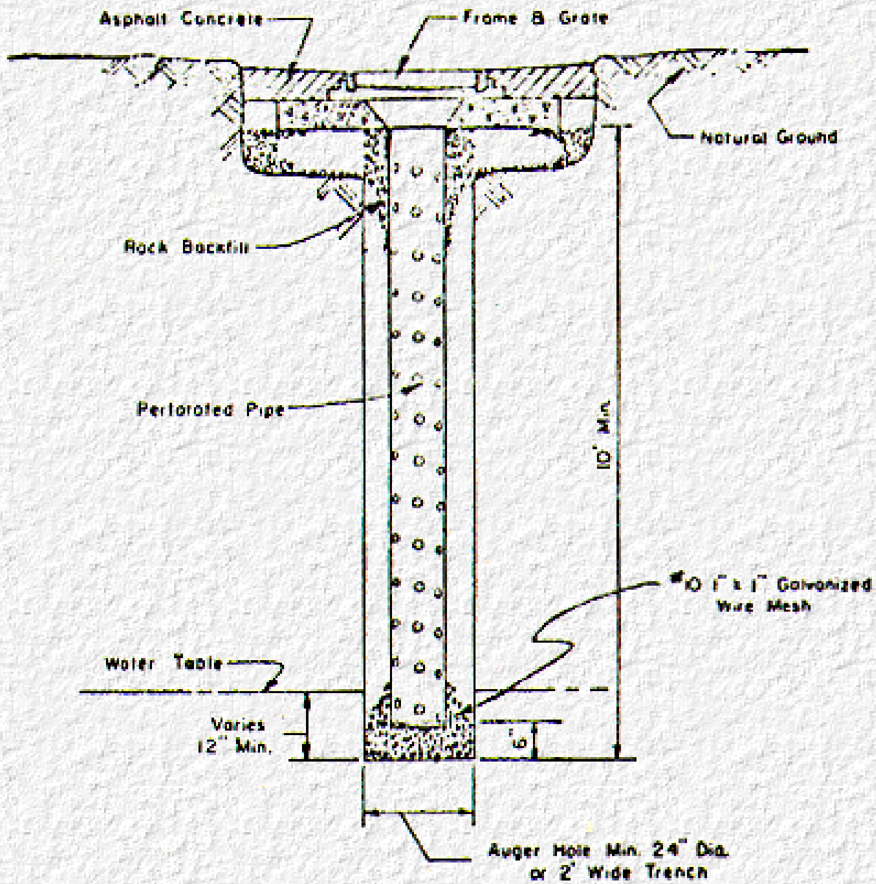


Figure 4-C-11. Drain Well with Drop Inlets. ***(Similar to Stanislaus County, Calif. Standard Drainage Unit).***



Note: Pipe and Rock Backfill Deleted When Field Conditions Permit Vertical Walls in Undisturbed Rock Areas.

Figure 4-C-12. Details for Catch Basin Auger Well. ***(Courtesy of Dade Country, Dept. of Public Works, Miami, Florida)***

The analysis of the site should be conducted in the same manner as that applied to other infiltration systems. In the site evaluation process, the biggest problem posed is the proper determination of infiltration drainage capacity. Several procedures for infiltration measurements are suggested in Chapter 4-A. The most reliable method of predicting well performance is through a falling or constant head infiltration test in augured holes.

When estimating well drainage rates, the effect of interference from other nearby wells should be considered. If drain wells are installed in groups, no individual well will have as large an infiltration capacity as it would have when isolated from the others. As groundwater builds up beneath each well, the outflow through every installation becomes partially restricted by the mounded water from neighboring wells. It is impossible to accurately predict a well's loss of drainage capacity due to mounding interference, but some lowering of individual well capacity must be expected when drain wells are clustered. The further apart these facilities are spaced, the less infiltration restriction there will be.

The first step in well design is the determination of the storm water inflow into the system. The inflow-time concept presented for trench design is also applicable for wells. From the hydrologic analysis, the requirements for storage volume and infiltration drainage capacity can be determined.

b. Example Problem 5

Vertical 36 inch (0.915 m) diameter drain wells have been considered for the correction of an isolated drainage problem that has developed as a result of subsidence in a small subdivision in Northern California. Field reconnaissance indicates that the surface drainage from about 1 acre (4047 m^2) contributes to the problem.

Assume that total runoff from this area must be contained by a system of vertical wells. Assume a 10-year frequency design storm using the rainfall intensity data from Table 4-C-3.

The contributing area consists of the following:

> 0.4 acres of paved streets, driveways and house roofs with runoff coefficient, $C_1 = 0.9$
>
> 0.6 acres of lawns with runoff coefficient, $C_2 = 0.2$
>
> Using the modified Rational Formula, V = CAIt
>
> $$CA = C_1A_1 + C_2A_2 = 0.9\,(0.4) + 0.2\,(0.6)$$
> $$= 0.36 + 0.12 = 0.48$$

Information is then developed for the mass inflow curve using the rainfall intensity values from Table 4-C-3.

The mass inflow curve is shown on Figure 4-C-13. Ponding is allowed to occur along the gutter for a length of 200 feet (61 m) to an average depth of 0.3 foot (0.092 m) at the gutter line and extending into the paved area a distance of 12 feet (3.66 m).

Volume of gutter storage = $\dfrac{(0.3)(12)(200)}{2}$ = 360 cu ft (10.08 m^3)

Table 4-C-3. Rainfall Intensity Values (I) Inches per Hour (10 Year Frequency) (3)

Min.	(I)	Min.	(I)	Min.	(I)	Min.	(I)	Min.	(I)
10	2.00								
11	1.93	21	1.42	31	1.15	41	0.95	51	0.84
12	1.86	22	1.38	32	1.13	42	0.93	52	0.83
13	1.79	23	1.34	33	1.11	43	0.92	53	0.82
14	1.73	24	1.31	34	1.09	44	0.91	54	0.81
15	1.68	25	1.28	35	1.07	45	0.90	55	0.80
16	1.63	26	1.25	36	1.05	46	0.89	56	0.79
17	1.58	27	1.23	37	1.03	47	0.88	57	0.78
18	1.54	28	1.21	38	1.01	48	0.87	78	0.77
19	1.50	29	1.19	39	0.99	49	0.86	79	0.76
20	1.46	30	1.17	40	0.97	50	0.85	60	0.75

* Minimum

	Minutes												
Hours	**0**	**5**	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**
1	0.75	.72	0.69	0.66	0.64	0.62	0.60	0.58	0.56	0.54	0.52	0.50	0.48
2	0.48	0.47	0.46	0.45	0.44	0.43	0.42	0.41	0.40	0.39	0.38	0.38	0.37
3	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.34	0.33	0.33	0.32	0.32	0.31
4	0.31												
**For intervals not shown use straight line interpolation													

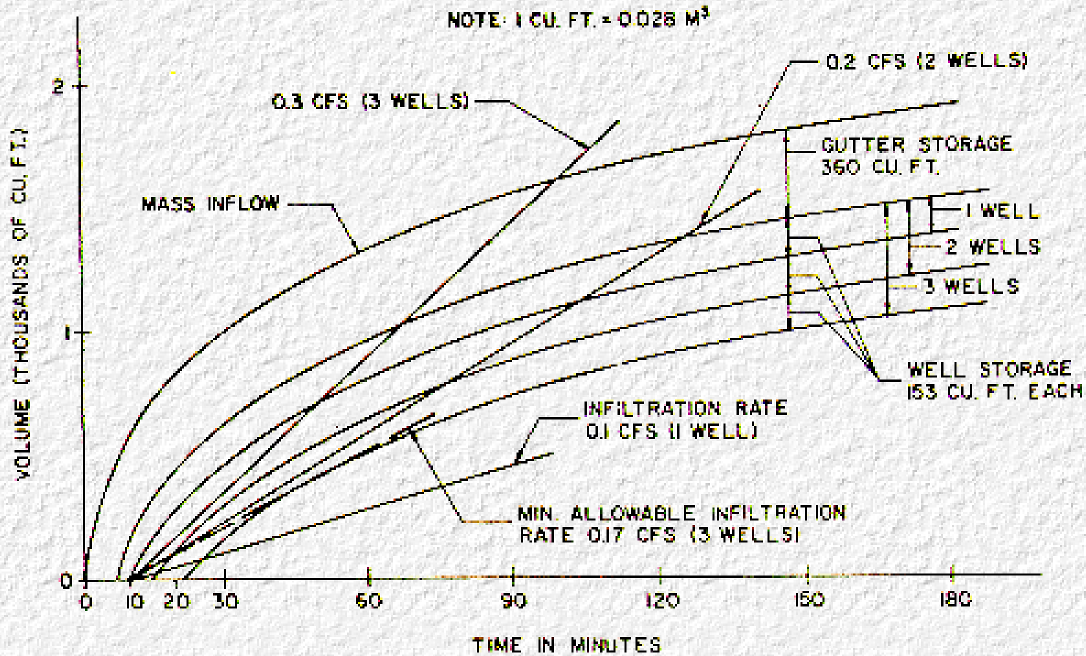


Figure 4-C-13. Mass Inflow, Storage and Cumulative Infiltration Rates for Well Design

Falling head infiltration tests made in two small auger holes to a depth of 30 feet (9.15 m) near the gutter line suggest an infiltration rate of 0.1 cu ft/sec (0.0028 m^3/sec) for a 36 inch (0.915 m) diameter well 30 feet (9.15 m) deep. The combined infiltration rates for 1, 2 and 3 wells are presented on Figure 4-C-13. The wells bottom out 10 feet (3.05 m) above the water table.

The storage capacity of a 36 inch (0.915 m) diameter well 30 feet (9.15 m) deep is determined as follows:

Assume 24 inch (0.61 m) diameter perforated pipe extending for full depth backfilled with aggregate material around the pipe consisting of sine number 467 from Table 4-C-3, 1 1/2-inch to No. 4 (37.5 to 4.75 mm). Assume 50 percent voids in backfill.

Pipe volume/ft = $\pi r^2(1) = \pi(1)^3$ = 3.14 cu ft/ft (0.288 m^3/m)

Voids in backfill = $[\pi(1.5)^2(1)-3.14]$ (0.5) = 1.96 cu ft/ft (0.180 m^3/m)

Total storage per well = (3.14 + 1.96) 30 = 153 cu ft (14.05 m^3)

The well storage for 1, 2 and 3 wells is shown on Figure 4-C-13.

Figure 4-C-13 allows for the selection of 1, 2, or 3 wells for the above example problem. In analyzing this graphical solution the line for infiltration rate must be above the storage volume for the well, i.e., 1 and 2 well designs are insufficient. Three vertical wells are required to correct this drainage problem with the data presented. The minimum allowable infiltration rate from Figure 4-C-13 for 3 wells is equivalent to 0.17 cfs. The safety factor is:

$$\text{S.F.} = \frac{\text{Available Infiltration Rate}}{\text{Minimum Allowable Infiltration Rate}} = \frac{0.3}{0.17} = 1.76$$

The design is satisfactory, but additional refinements can be made.

This example illustrates one of several approaches that can be used to size vertical drainage wells. In most cases, well design will be governed by local procedures and experience.

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5. Design of Filter Systems

Filter systems for infiltration drainage applications must be capable of filtering out fine materials suspended in storm water to prevent clogging of native soil, which would reduce its infiltration capacity. The filter would also serve to prevent the migration of fines from the native soil into the drainage layer of the infiltration system, i.e., pipe backfill. Such migration could occur during periods of high groundwater which can create reverse flow conditions.

The following gradation criteria, established from work by Terzaghi, Bertrum, and the Corps of Engineers, is suggested for design of filter and drain systems:

Drains that are to protect work from water damage require pore spaces small enough to trap the adjacent soils and prevent their movement into or through the drain, and yet large enough to allow water to enter the drain without excessive head build-up. In order to prevent migration of fines and clogging, the following criterion is generally used:

$$\frac{D_{15}(\text{of filter})}{D_{85}(\text{of soil})} \leq 5 \qquad (4\text{-C-}1)$$

And to insure that a filter or drain will be somewhat more permeable than an adjacent finer filter or soil, the following criterion was suggested by Terzaghi:

$$\frac{D_{15}(\text{of filter})}{D_{15}(\text{of soil})} \geq 5 \qquad (4\text{-C-}2)$$

In these equations, D_{15} designates the 15% size of a material, and D_{85} the 85% size (15% is finer or 85% is finer).

To insure that the gradation curve of a filter aggregate will be somewhat parallel to the curve for a soil, the following requirement is often used:

$$\frac{D_{50}(\text{of filter})}{D_{50}(\text{of soil})} \leq 25 \qquad \text{(4-C-3)}$$

To prevent movement of filter or drain aggregate through slots in pipes the U. S. Army Corps of Engineers uses the following criterion for gradation of filter material in relation to the sizes of slots:

$$\frac{\text{85\% size of filter material}}{\text{slot width}} > 1.2 \qquad \text{(4-C-4)}$$

And for circular holes the Corps uses the following criterion:

$$\frac{\text{85\% size of filter material}}{\text{hole diameter}} > 1.0 \qquad \text{(4-C-5)}$$

Cedergren(4) emphasizes that Equation 4-C-2, while generally assuring that a filter material will be perhaps 15 to 25 times more permeable than a protected soil, does not always insure sufficient permeability in filters or drains. In many situations, a much higher level of permeability is needed. In general. if the flow is across the thin dimensions of a filter and into a highly pervious zone or drain that provides the necessary water removal, Equation 4-C-2 will assure that the filter is more permeable than a soil being drained and head losses in the filter are not likely to be excessive. But, if the flow is alone the thin dimension of a drain and allowable hydraulic gradient is of a limited value, say 0.01 to 0.05 ft/ft, , it is necessary to determine the permeability needs of the drain by making an appropriate seepage analysis. For further information refer to a reliable reference text on the subject.

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6. Filter Cloth

Filter fabrics and cloth, referred to as geotextiles, are used to prevent mixing of fine soils with high porosity drainage aggregates in underground disposal systems.

The choice of filter cloth or fabric is dependent on the size and nature of the fines. Filter cloth must have sufficient permeability to prevent constricting flow. The "Equivalent Opening Size", or EOS, of the fabric should be balanced with the soil permeability. Refer to Appendix E-1 for method of test to determine EOS. Drainage experts or soil mechanics engineers should be consulted on the selection of a particular fabric. Refer to Appendix E-2 for filter fabric specifications.

The most widely accepted modern theory of filtration for drainage type applications is that of aggregate bridging, i.e., filtration is established by successive bridging of particles across pores which are provided when larger graded aggregate is in contact with relatively smaller graded aggregate. While the voids in the larger graded aggregate may be substantial and large enough to pass single particles through the system, multiple particles when in contact with each other and the larger graded material will bridge this gap, maintain permeability due to the bridging action and minimize hydraulic piping. This system is referred to as a "filter cake". It is this cake development which provides the filtration whether it is for mineral aggregate filtration or for filter fabrics.

This modern theory is consistent with the gradation criteria established for mineral aggregates as defined in Section 5 of this chapter, "Design of Filter Systems".

The multiple layer bridging network of mineral aggregate filters is necessary because of the disassociation of aggregates when overly large pore spaces are available, i.e., the fine graded mineral aggregates will pipe through the voids of very coarse graded mineral aggregates. The structural integrity of the man-made filter media or geotextile will not, however, disassociate as do the graded aggregate layers. Therefore, it can be installed in a single layer with coarse graded rock within, i.e., trench backfill.

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7. Natural Filters

Natural filters such as grass and other vegetation or straw bales can be utilized around drainage inlets in unpaved areas to detain, absorb, and filter surface runoff and remove sediment from the water. These vegetative buffers or filters can significantly reduce the quantity of sediment entering the infiltration system. Flattening the slopes in the buffer area will further reduce potential sediment pollution.

References

1. Practices in Detention of Urban Storm Water Runoff, American Public Works Association, Special Report No. 43, 1974.
2. Weaver, R. J., "Recharge Basins for Disposal of Highway Storm Drainage", Engineering Research and Development Bureau, New York DOT, Research Report 69-2, May 1971.
3. Manual for Storm Drainage Design, San Joaquin County, California, Department of Public Works, January 1973.
4. Cedergren, H. R., Seepage, Drainage, and Flow Nets, 2nd Ed., John Wiley and Sons, Inc., New York, pp. l90-207, 1977.

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Construction Methods and Precautions

Go to Chapter 6

A. General

Regardless of the type of infiltration system to be constructed, careful consideration should be given in advance of construction to the effects the work sequence, techniques, and equipment employed will have on future maintenance of the system. Serious maintenance problems can be averted, or in large part mitigated, by the adoption of relatively simple measures during construction.

B. Basin Construction

The sequence of various phases of basin construction has an identifiable relationship to the overall project construction schedule. An ideal program would schedule rough excavation of the basin for the rough grading phase of the project to permit use of the material as fill in earthwork areas. The partially excavated basin could serve as a sedimentation basin to assist in pollution control during construction. However, basins near final stages of excavation should never be utilized prematurely for runoff disposal. Drainage from untreated, freshly constructed slopes within the watershed area would load the newly formed basin with a heavy concentration of fine sediment. This could seriously impair the natural infiltration characteristics of the site.

Specifications for basin construction should state: (1) the earliest point in progress when storm drainage may be directed to the basin; and (2) the means by which this delay in opening is to be accomplished. Due to the wide variety of conditions encountered among projects, each should be separately evaluated in order to postpone opening as long as is reasonably possible.

Establishment of turf on the basin side slopes and floor is recommended. A dense stand of turf will not only prevent erosion and sloughing, but will also provide a natural means of maintaining relatively high infiltration rates through the surface. Removal of accumulated sediment is a problem only at the basin floor, when the basin is adequately maintained. Little, if any, additional maintenance is normally required to maintain the infiltration capacity of the slope areas.

Initial basin excavation should be carried to within 1 ft (0.3 m) of the final elevation of the basin floor. Final excavation to the finished grade should be deferred until all slopes in the watershed have been seeded and protected with an interim treatment. The final phase excavation should be performed carefully to remove-all accumulated sediment. Relatively light equipment is recommended for this operation to avoid deep compaction of the basin floor. After the final grading is completed, the basin floor should be deeply tilled by means of rotary tillers or disc harrows to open the soil pores and provide a well-aerated, highly porous surface texture(1). The use of dual purpose sedimentation-infiltration basins is dependent upon the potential silt load in the storm runoff, after project construction. It is recommended that such basins be left incomplete with undeveloped ground cover on the basin floor for 1 to 2 years after construction to permit silt accumulation to stabilize. Sediment accumulated may have to be removed several times during and after construction and the basin floor may require tilling to restore infiltration capacity.

When coarse organic material are specified for discing (such as cotton boll hulls, leaves, stems, etc.) or spading into the basin floor to increase the permeability of clay soils, the basin floor should be soaked or inundated for a brief period, then allowed to dry subsequent to this operation. This induces the organic material to decay rapidly, loosening the upper soil layer(2). For design information on infiltration basins refer to Chapter 4-C.

C. Trench Construction

Trench construction techniques will vary with local soil conditions. Designs may consist of either slab-covered trenches, devoid of both backfill and drainage conduits (Figure 5-1); or trenches constructed using permeable backfill and perforated or slotted pipe (Figure 5-2). For detailed design requirements refer to Chapter 5-C.

Infiltration or recharge trenches can be constructed with minimal difficulty in various types of stable soil or rock deposits where sufficient permeability for drainage is provided. Strict adherence to OSHA's Trench Safety Code, or other local regulations relative to acceptable construction practice, should be observed.

Depending upon the length and width of trench, either a backhoe or wheel or ladder type trencher may be used to excavate a 3 to 4 ft (0.9 to 1.2 m) trench. Rock should be rippable since costs become prohibitive when blasting is required.

Figure 5-1. Typical Slab-Covered Infiltration Trench in Pervious Rock*. (Courtesy of Dade*

County, Dept. of Public Works, Miami, Florida)

Figure 5-2. Typical Infiltration Trench with Perforated Pipe and Permeable Backfill. ***(Courtesy of Dade County, Dept. of Public Works, Miami, Florida)***

When a trench is excavated in rock having adequate strength to be self-supporting under a specified wheel load (as defined in Chapter 4-C), it may be covered with slabs of concrete, steel, or aluminum, without need of either backfill or drainage conduits. Figures 5-3 through 5-6 show typical construction operations for both precast and cast-in-place slab-covered trenches.

When a trench with perforated or slotted pipe is to be constructed using free-draining coarse aggregate backfill, the backfill is placed in the bottom of the trench and brought up to the pipe flowline elevation (Figure 5-7). The pipe is then placed and covered with a minimum of 1 ft (0.3 m) of coarse aggregate backfill. Where silts or other fine-grained materials are present in the trench excavation, filter cloth or fabric placement is recommended to deter backflow of "fines" into the coarse aggregate or permeable backfill material during periods of high groundwater. The filter cloth is used to encase the aggregate backfill and conduit as shown in Figure 5-7 and Figure 5-8. An impermeable barrier is also required over the top of backfill to prevent the vertical infiltration of silts and sediments into the backfill and prevent possible subsidence (Figure 5-9). This barrier could consist of two layers of 30 lb construction-quality felt, or two layers of construction polyethylene sheeting(3). Following the placement of this impervious barrier, the trench is backfilled with native soil and/or pavement as required to conform with surface grade.

Figure 5-3. Excavation for Slab-Covered Trench. ***(Caltrans Photo, Miami Area*****)**

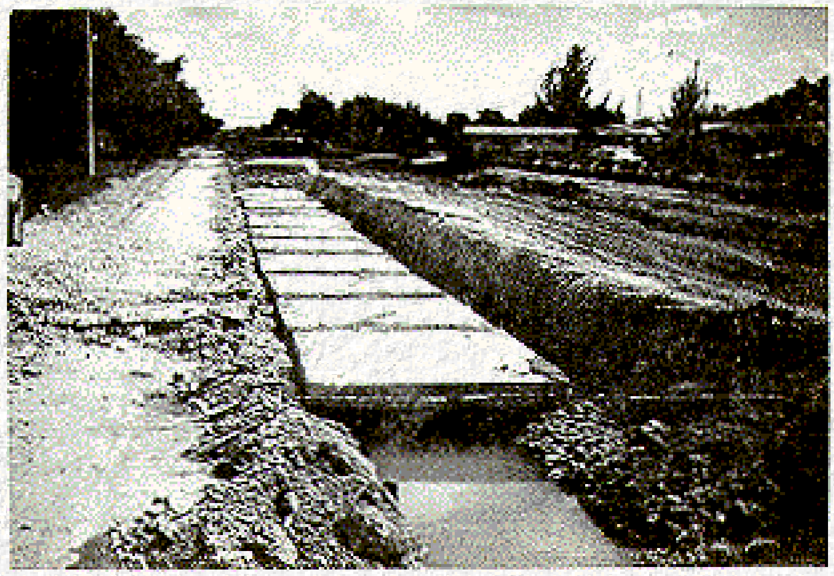

Figure 5-4. Precast Concrete Slabs for Covered Trench. ***(Caltrans Photo, Miami Area)***

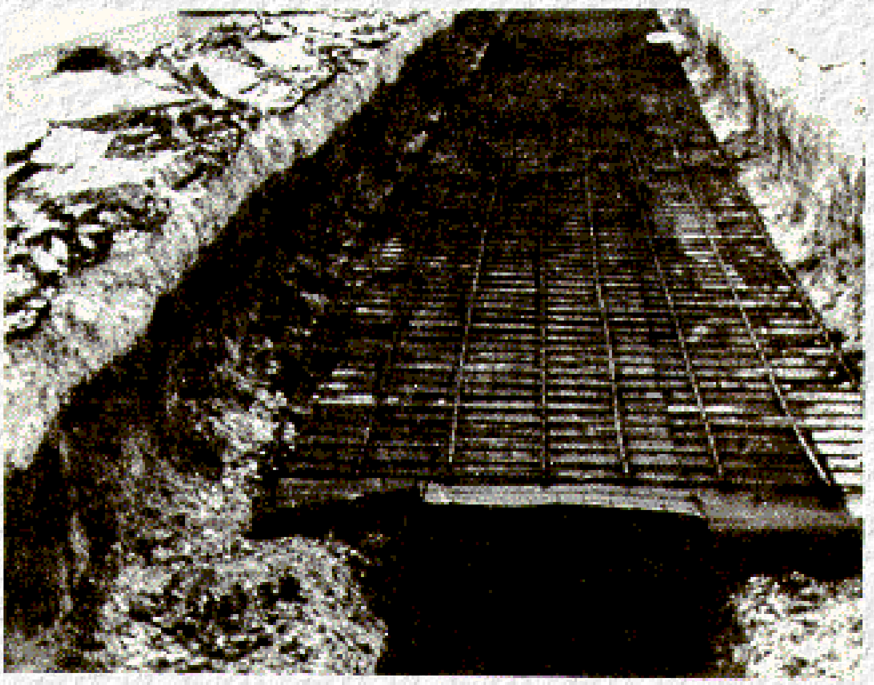

Figure 5-5. Placement of Reinforcement for Cast-In-Place Concrete Slab-covered Trench. ***(Courtesy of Dade County, Dept. of Public Works, Miami, Florida)***

Figure 5-6. Concrete Slab-Cover in Place, Note Groundwater Level in Excavation. ***(Courtesy of Dade County, Dept. of Public Works, Miami, Florida)***

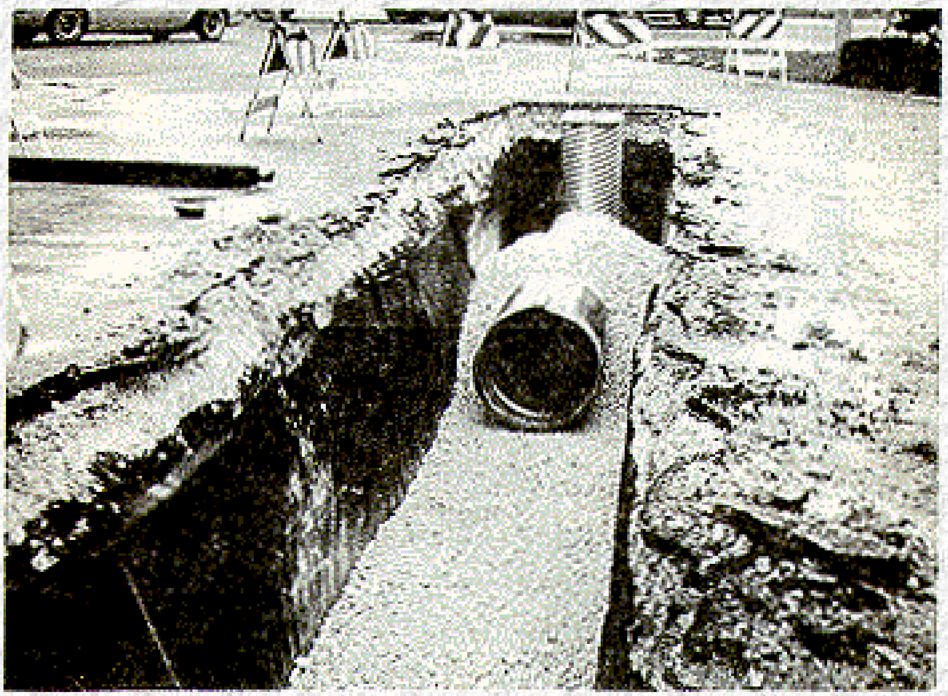

Figure 5-7. Trench Backfilled to Flow Line Grade with Filter Cloth on Side Slope

Figure 5-8. Placement of Perforated Pipe in Trench with Aggregate Backfill and Filter Cloth

Figure 5-9. Impermeable Barrier of Builders Felt Covering Trench Backfill.
(Courtesy of Dade County, Dept. of Public Works, Miami, Florida)

The trenching operation should be performed in assembly line fashion. The rock is placed directly after excavation, followed by laying of pipe and placing of the felt and native soil. Care should be taken to excavate only that amount of trench that Can be completed within one work-day. If too much trench is excavated and the walls collapse, the trench can be reexcavated; but the failed wall areas will have to be replaced with aggregate, increasing construction costs. If rock is at a premium, such cost increases could become considerable. Rainfall during the construction can also lead to the influx of sediments into the excavation, resulting in clogging of the pervious layers of the trench wall. Consideration should be given to scarification of the trench floor and walls prior to backfilling to restore infiltration capacity diminished by sediment deposit during storm inflow or by smearing as a result of construction. Care must also be taken to ensure that perforated pipe is not placed within 10 ft (3.1 m) of structures where piping could occur and result in surface subsidence.

One method (employed with some success) for reducing the quantity of costly backfill involves use of a slip form of either plywood or steel. The top and bottom of the form are left open and it is so designed that a small crane or backhoe can lift the form with cables. To prevent fine soil particles from being drawn into the aggregate backfill, causing pavement settlement, a porous filter cloth is wrapped and enveloped around the aggregate backfill as discussed above. After excavation, the slipform is set in place, the filter cloth laid over the sides of the form and the aggregate backfill is placed on the bottom of the trench, within the form walls. As native fill material is pushed in from each side of the trench, the form is lifted and the fill holds the rock in place. This operation is continued until the system is completed to the top of the aggregate backfill. The filter cloth is then laid over the top of the aggregate backfill to finish the trench. The use of the filter cloth obviates the necessity of using filter material, as the cloth will pass water but not fine soil particles. The remainder of this system is completed as discussed above.

D. Well Construction

The choices for drilling equipment, well casing, and well construction design depend on the intended use in addition to geologic and economic factors. Wells can be either cased or uncased, or consist of vertical shafts backfilled with free draining material(2,4,5). For details on well design refer to Chapter 4-C.

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1. Shallow Wells

Shallow wells are usually less than 25 feet (7.63 m) deep and normally are angered by either small-diameter 8 to 12 inches (0.2 to 0.305 m) flight auger drilling, spin augers 9 to 30 inches (0.23 to 0.76 m) in diameter, or by rotary bucket augers of up to 3 feet (0.9 m) or more in diameter. Rotary bucket drilling is the most commonly used well boring method (refer to Figure 5-10). Rotary bucket drilling has application primarily in areas with clay formations that will stand without caving until the borehole is completed. Cobbles and boulders cause difficulty and must be removed by special devices to permit continuation of the hole. The average rotary bucket rig has a depth capability of about 75 feet (23 m). Small-diameter auger flight drilling is the fastest and least expensive in soil (Figure 5-2). Rotary drilling in firmer deposits can be accomplished using a rotary pilot hole and hole opener bit.

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2. Deep-Wells

Wells deeper than 25 feet (7.63 m) are considered deep wells and their construction employs various techniques that are utilized in the water well drilling industry, as discussed below.

Figure 5-10. Rotary Bucket Drilling for Well Construction. ***(Courtesy of Caltrans)***

Figure 5-11. Small-Diameter Auger Flight Drilling for Well Construction. ***(Courtesy of Caltrans)***

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a Hydraulic Rotary Drilling

Conventional rotary drilling is generally the most rapid and economical method for drilling deep wells in unconsolidated formations. The benefits of this procedure increase with drilling depth. Small pilot holes can be drilled, and gravel-packed wells are easily constructed. Some of the disadvantages of this procedure are that the depth to static water level cannot be accurately measured, and precise logging of cuttings is often difficult. Boulders can also present difficulty.

When drilling mud is used to keep the hole open during the drilling operation, care should be taken to flush the hole to remove all mud before backfilling. Drilling muds can cause side wall caking, which impedes infiltration. If the drilling mud used is of the degradable type flushing will probably not be required. Careful selection of a drilling mud is important as certain muds may promote the growth of a coliform bacteria and are therefore environmentally unacceptable.

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b. Cable-Tool Drilling

Cable-tool (or percussion method) drilling is normally preferred when drilling to moderate depths in unconsolidated or consolidated formations. Difficulty occurs in caving formations. Sampling unconsolidated materials by this method usually presents relatively few difficulties, and only a small amount of water is required for the drilling operation. This procedure can be used for gravel-packed wells. Negative aspects include the cost of casing and the need for careful reconstruction of the original in situ depositional pattern when logging formational cuttings. The method is considered slow, and there are limitations on achievable depth.

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c. Reverse Circulation

Reverse circulation is a technique employed to overcome mud removal problems in unconsolidated materials. It can be used to drill holes of large diameter, from an 18-inch (0.46 m) minimum up to 60 inches (1.53 m). The hole cannot be kept open for longer than about 36 hours. Drilling is accomplished without air to 450 ft (137 m) in depth and with air to 1000 ft (305 m) in depth. A large water supply and head from 6 to 30 ft (1.8 to 9.2 m) is required to prevent excessive fluid loss. The method is relatively fast and is generally used for gravel packed wells.

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d. Air Rotary Drilling

Air rotary is the same as straight rotary except air is the drilling fluid. The method has depth limitations and is used only on consolidated materials. The overburden materials require casing.

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e. Down-the-Hole

This method consists of pneumatically-operated bottomhole drilling combining percussion and turning action, is the fastest method in hard rock materials. Drilling can be accomplished with 6 to 8-inch (0.15 to 0.20 m) diameters to depths of 1000 feet (305 m).

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f. Other Methods

Other methods are available, but their application is limited to small diameter wells.

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3. Well Developments

This procedure corrects the damage to, or clogging of, permeable formations that results as a side effect of drilling. Its purpose is to increase the porosity and permeability of the natural formations in the vicinity of the well. Well development also stabilizes the sand formations around a screened well, bringing into the well designed amount of material to effect the desired packing. Methods used vary with type of construction and equipment available. Jetting or surging are two procedures employed in well development.

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Maintenance and Inspection

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A. General

Drainage systems should be inspected on a routine basis to ensure that they are functioning properly. Inspections can be on an annual or semi-annual basis, but should always be conducted following major storms. Systems that incorporate infiltration are most critical since poor maintenance practices can soon render them inefficient. Inspection of pipes, covered trenches, and wells can be accomplished with closed circuit television; and still photographs can be obtained by either taking a picture of the monitor, or mounting a still camera alongside the T.V. camera and triggering it electronically. Other more economical alternate methods of inspection are also available. Procedures for maintenance of these systems are discussed in this chapter. It should be stressed that good records should be kept on all maintenance operations to help plan future work and identify facilities requiring attention.

B. Basins

Infiltration basin surfaces are sometimes scarified to break up silt deposits and restore topsoil porosity. This should be accomplished after all sediment has been removed from the basin floor. However, this operation can be eliminated by the establishment of grass cover on the basin floor and slopes. Such cover helps maintain soil porosity.

Algae or bacterial growth can inhibit infiltration. While chlorination of the runoff water can solve this problem, it is more practical to make certain that the basin is permitted to dry out between storms and during summer months.

Holding ponds or sedimentation basins can be used to reduce maintenance in conjunction with infiltration basins by settling out suspended solids before the water is released into the infiltration basins.

Chemical flocculants can be used to speed up settlement in holding ponds. Flocculants should be added to the runoff water within the settlement pond inlet pipe or culvert where turbulence will ensure more thorough mixing. After suspended matter has flocculated and settled in the pond, the water may be released into the infiltration basin for disposal. Although chemical flocculants may be impractical for general use, they might well be considered in special cases.

Alum (Aluminum Sulfate) is readily available, inexpensive, and highly effective as a flocculating agent. It is widely used in water treatment plants. Various trade name flocculation agents are also available.

Cleanout frequency of infiltration basins will depend on whether they are vegetated or non-vegetated and will be a function of their storage capacity, infiltration characteristics, volume of inflow, and sediment load. Infiltration basins should be inspected at least once a year. Sedimentation basins and traps may require more frequent inspection and cleanout.

Grass bottoms on infiltration basins seldom need replacement since grass serves as a good filter material. This is particularly true of Bermuda grass, which is extremely hardy and can withstand several days of submergence. If silty water is allowed to trickle through Bermuda, most of the suspended material is strained out within a few yards of surface travel. Well-established Bermuda on a basin floor will grow up through silt deposits, forming a porous turf and preventing the formation of an impermeable layer. Bermuda grass filtration would work well with long, narrow, shoulder-type (scales, ditches, etc.) depressions where highway runoff flows down a grassy slope between the roadway and the basin. Bermuda demands very little attention besides summer irrigation and looks attractive when trimmed. Planted on basin sideslopes it will also prevent erosion.

Non-vegetated basins can be scarified on an annual basis following removal of all accumulated sediments. Rotary tillers or disc harrows with light tractors are recommended for maintenance of infiltration basins where grass cover has not been established. Use of heavy equipment should be discouraged to prevent excessive compaction of surface soils. The basin floor should be left level and smooth after the tilling operation to ease future removal of sediment and minimize the amount of material to be removed during future cleaning operations. A levelling drag, towed behind the equipment on the last pass, will accomplish this.

Coarse rock or pea gravel is often placed on the bottom of a drainage basin to prevent the formation of a filter cake on the soil, by screening out suspended solids. After a period of operation the aggregate becomes partially clogged, and it is then necessary to remove and clean it, or replace it with new material. This could be accomplished on an annual basis. Inasmuch as basins are usually accessible, this kind of operation is seldom expensive or difficult. The disposal of silt and other sediments should comply with local area codes.

C. Trenches

The clogging mechanism of trenches is similar to that associated with other infiltration systems. Although the clogging of trenches due to silt and suspended material is more critical than that of basins, it is less critical than the clogging of vertical wells. The use of perforated pipe will minimize clogging by providing catchment for sediment without reducing overall efficiency. Maintenance methods associated with these systems are discussed later in this chapter.

D. Wells

The same clogging and chemical reactions that retard basin and trench infiltration will affect wells, to an even greater extent. One problem unique to wells is chemical encrustation of the casing, with consequent blocking of the perforations or slots in the well casing. Alternate wetting and drying builds up a scale of water-soluble minerals, which can be broken up or dissolved by jetting, acid treatments, or other procedures.

Some agencies restore well efficiency by periodic jetting, which removes silt and fines. Jetting consists of partially filling a well with water, then injecting compressed air through a nozzle placed near the bottom of the shaft. (Refer to Section F-4 of this chapter.) Dirt or sand that has settled in the shaft or has clogged the casing perforations is forced out the top of the well. Wells cleaned in this manner will operate fairly efficiently for several years, providing that drainage was good initially.

Clogging due to silt and suspended material is much more critical in cased wells than in basins. Filters or sedimentation basins and special maintenance procedures will help prevent silting up of wells. Underground sediment traps in the form of drop inlets are frequently used with small wells, but these inlets do little more than trap the heaviest dirt and trash, allowing finer suspended matter to flow into the well. Larger settling basins hold water longer for more efficient silt removal, and provide some temporary storage volume.

Sand and gravels or other specially-selected filter materials used in "gravel packed" wells cannot be removed for cleaning if they should become clogged. Well screens that become partially or totally clogged by corrosion, bacteria, or other deposits are also not removable for repair. Generally, the only practical solution to the problem is to drill another well and abandon the inoperative one. Problems of clogging of gravel packing (and well walls) can often be minimized by using sediment traps and by treating the water to remove substances that will clog the soil, the gravel packing, or the well screen. Problems of corrosion of well screens can be eliminated by using slotted PVC pipes for well screens. Furthermore, PVC resists attack by acids or other chemicals sometimes used for flushing wells to remove deposits that clog the gravel packing or the well walls.

It is important that those maintaining infiltration facilities that employ wells be knowledgeable of the kind of materials used in screens and other parts of the systems that are vulnerable to damage by acids and other corrosive substances. The importance of regular well maintenance cannot be overstressed. Periodic cleaning and redevelopment is essential, and chlorination or other chemical treatments may be necessary if biological growth or encrustation impedes drainage. Should there be any signs of bacterial groundwater contamination, a 5-10 ppm dosage of chlorine should be added to the wells in question.

During the design stage of infiltration well systems consideration should be given, to the maximum extent practicable, to the use of filter materials that would facilitate maintenance. If aggregate filter material is mounded over an infiltration well as shown in Figure 6-1, designers should realize that it will be necessary to periodically remove the upper part of the filter material and clean it or replace it with clean material. In some situations this may not be practical. When cased, gravel-packed wells are used it would be impractical to use a fine aggregate filter, although some designers make use of a bag constructed of filter fabric fitted to the top of a well to trap sediment. When the inflow rate has decreased the maximum tolerable amount, the bag is removed and cleaned much as a vacuum cleaner bag is cleaned; or a new filter bag is inserted. Consideration should also be given to backflushing the well system using methods similar to those defined in Sections F-4 and F-5 of this chapter.

E. Catch Basins

Catch basins should be inspected after major storms and be cleaned as often as needed. Various techniques and equipment are available for maintenance of catch basins, as discussed under Section F. Filter bags can be used in catch basins at street grade to reduce the frequency for cleaning catch basins and outflow lines. Filter bags similar to that shown in Figure 6-2 have been used successfully in Canada and various parts of the United States.

F. Methods and Equipment for Cleanout of Systems

Various types of equipment are available commercially for maintenance of infiltration systems. The most frequently used equipment and techniques are listed below.

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1. Vacuum Pump

This device is normally used to remove sediment from sumps and pipes. The equipment for this system is generally mounted on a vehicle. It requires a 200 to 300 gallon (0.757 to 1.136 m^3) holding tank and a vacuum pump that has a 10-inch (254 mm) diameter flexible hose with a serrated metal end for breaking up cake sediment. A two-man crew can clean a catch basin in 5 to 10 minutes. This system can remove stones, bricks, leaves, litter, and sediment deposits. Normal working depth is 0 to 20 feet (0 to 6 m).

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2. Waterjet Spray

This equipment is generally mounted on a self-contained vehicle with a high pressure pump and a 200 to 300 gallon (0.760 to 1.140 m^3) water supply. A 3-inch (76 mm)flexible hose line with a metal nozzle that directs jets of water out in front is used to loosen debris in pipes or trenches. The nozzle can also emit umbrella-like jets of water at a reverse angle, which propels the nozzle forward while blasting debris backwards toward the catch basin. As the hose line is reeled in, the jetting action forces all debris to the catch basin where it is removed by the vacuum pump equipment. Normal length of hose is approximately 200 feet (61 m). Because of the energy supplied by the water jet, it should not be used to clean erodible trench walls.

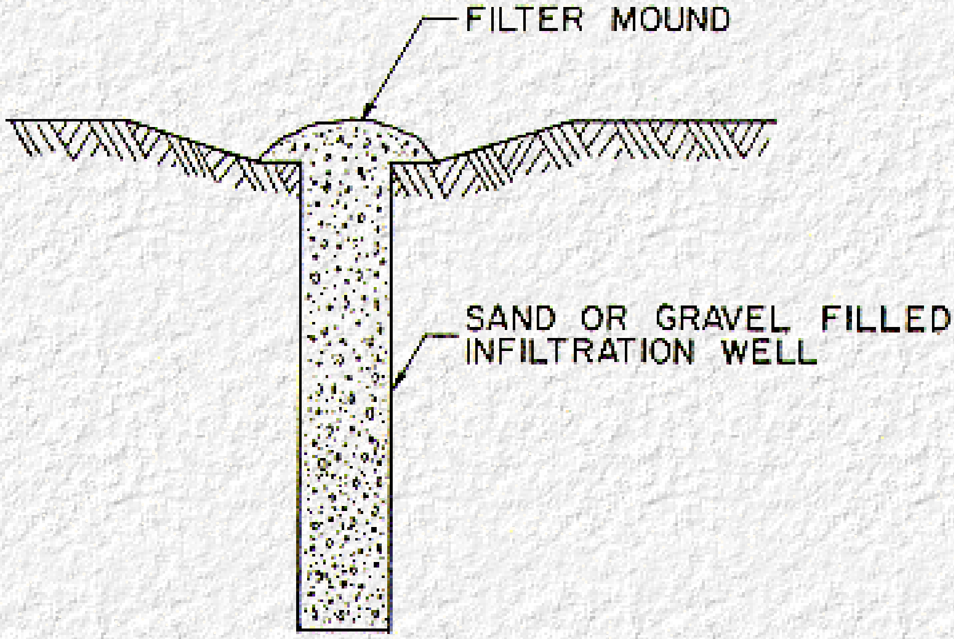

Figure 6-1. Typical Infiltration Well Covered with Filter Mound. ***(Courtesy of Caltrans)***

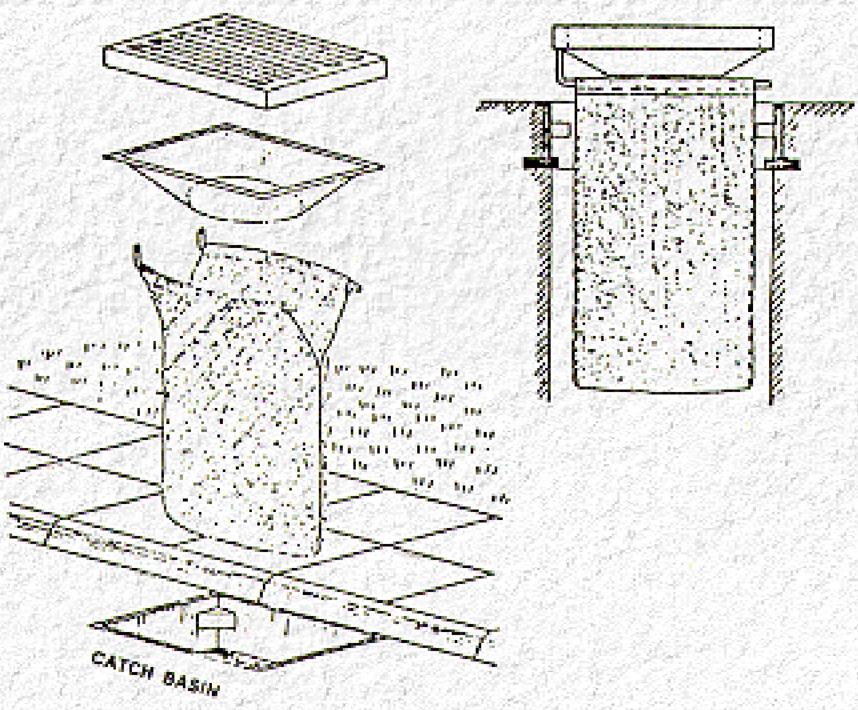

Figure 6-2. Filter Bag Application for Catch Basins*. (Courtesy of Hydro-Storm Sewage Corp., New York)*

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3. Bucket Line

Bucket lines are used to remove sediment and debris from pipes or trenches over 48-inch (1.22 m) in diameter or width. This equipment is the most common type available. The machine employs a gasoline engine-driven winch drum, capable of holding 1,000 feet (305 m) of 1/2-inch (13 mm) wire cable. A clutch and transmission assembly permits the drum to revolve in a forward or reverse direction, or to run free. The bucket is elongated; with a clam shell type bottom which opens to allow the material to be dumped after removal.

Buckets of various size are available. The machines are trailer-mounted, usually with three wheels; and are moved in tandem from site to site. When a length of pipe or trench is to be cleaned, two machines are used. The machines are set up over adjacent manholes. The bucket is secured to the cables from each machine and is pulled back and forth through the section until the system is clean. Generally when the bucket comes to the downstream manhole, it is brought to the surface and emptied.

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4. Compressed Air Jet

The compressed air jet is normally used to clean and remove debris from vertical wells. This equipment (Figure 6-3) requires a holding tank for the water and debris removed, a source of water supply (if the well is above the groundwater level), an air compressor, two 1/4-inch (6.4 mm) air lines, a diffusion chamber, and a 4-inch (102 mm) diameter pipe to carry the silty water and other debris to the ground surface. The well is partially filled with water, if required, and the compressed air injected through a nozzle near the bottom of the well. As the silty water enters the diffusion chamber (to which the other air line is connected) it becomes filled with entrained air and is forced up the 4-inch (102 mm) disposal pipe and out of the top of the well by the denser water entering the bottom of the diffusion chamber intake. Normal working depths are 0 to 75 feet (0 to 22.9 m).

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5. Surging and Pumping

This procedure is another means of removing silt and redeveloping a well. The process involves partially filling the well with water and then pumping a snug-fitting plunger up and down within the casing. This action loosens silt and sediment lodged in the packing and immediately adjacent soil, and pulls it into the well. Surging is immediately followed by pumping silt-laden water from the bottom of the well. If the well is situated in clayey soil or if clay materials have been washed into the well, the surging and air jetting methods will be more effective if sodium polyphosphate is added to the water in the well prior to cleaning or redeveloping. A 2-5 ppm concentration of this chemical will deflocculate clay particles in the well and immediately surrounding soil, and the clay can be pumped or jetted out very easily. Depth is limited by the pumping capacity available.

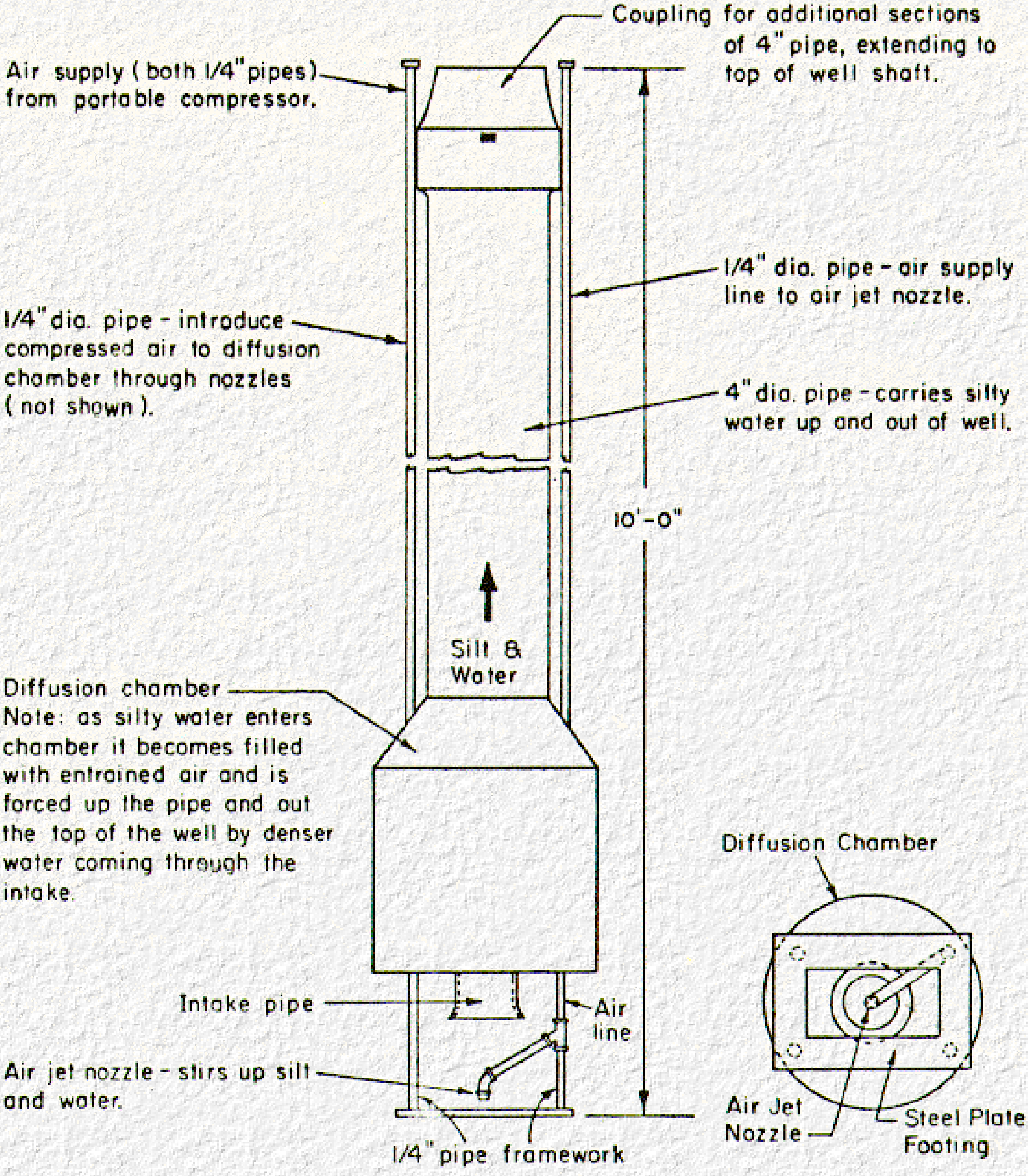

Figure 6-3. Compressed-Air Jet Cleaner. ***(Courtesy of Caltrans)***

6. Fire Hose Flushing

This equipment consists of various fittings that can be placed on the end of a fire hose such as rotating nozzles, rotating cutters, etc. When this equipment is dragged through a pipe, it can be effective in removing light material from walls.

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7. Sewer Jet Flushers

Sewer jet flushers are usually truck-mounted and consists of a large watertank of at least 1000 gallons (3.785 m^3), a triple action water pump capable of producing 1000 psi (6900 KN/m^2) or more pressure, a gasoline motor to run the pump, a hose reel large enough for 500 feet (153 m) of 1inch (25 mm) inside diameter high pressure hose, and a hydraulic pump to operate the hose reel. In order to clean pipes properly a minimum nozzle pressure of 600 psi (4140 KN/m^2) is required. All material is flushed ahead of the nozzle by spray action. This extremely mobile machine can be used for cleaning areas with light grease problems, sand and gravel infiltration, and for general cleaning.

References

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Appendix A : FHWA-TS-80-218

Summary of Key Information Obtained from Questionnaire on Infiltration Drainage Practices

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Agency or Firm	Utilization of Infiltration Drainage		Type of Systems Employed	Methods used to Determine Infiltration Rate	Frequency of System Maintenance	Remarks
	Yes	No				
Alabama Highway Dept.		X		Constant and falling head (U.S. Corps of Engrs)		
Arizona DOT		X				Retention Systems only.
Colorado Dept. of Highways		X				Retention Systems only.
Connecticut DOT	X		Dry wells[1]	Visual Inspection and Test Pits		Environmental Consideration - Possible contamination of water supply aquifers by salt used during winter for snow removal.
Delaware Div. of Highways	X		Vertical wells and trenches backfilled with sand	Grain Size Analysis	After each storm	Recharge not considered in design.

Florida DOT	X		Trenches and basins	Various procedures using theoretical and actual tests		Perforated pipe and course aggregate backfill used in trench construction. Use infiltration systems as a method of preventing pollutants from discharging into surface waters.
Georgia DOT		X				
Hawaii DOT	X		Vertical wells			Dynamite used to relieve clogging.
Idaho Transportation Dept.		X	Roof drain system used at State Laboratory	Laboratory-constant head and falling head. Field-Pump in or out, most often bail and recovery.		Environmental and legal restrictions in highly residential areas.
Illinois DOT		X		Laboratory falling head test with clay soils	Function of storage capacity and volume of runoff.	
Indiana State Hwy. Commission		X				
Iowa DOT		X				
Kansas DOT		X				
Maryland DOT		X	Vertical pit at one location	Laboratory falling head test with clay soils		Use of deicing salts prevent use of infiltration due to possible pollution of groundwater.
Massachusetts Dept of Public Works		X				

Michigan Dept. of State Hwys and Trans.		X		Laboratory-constant head and falling head. Field-Standard percolation test.		Extensive use of deicing salts percent use of infiltration drainage.
Minnesota DOT		X				State Pollution Control regulations prevent the use of infiltration systems for transmitting surface water runoff into aquifer.
Mississippi State Hwy Dept.		X				
Missouri State Hwy. Commission		X				
Nevada Dept of Hwys		X		Laboratory-Falling head test and grain size analysis. Field-Standard percolation test		
New Hampshire Dept of Public Works and Hwys		X		Visual inspection and soil classification	6 months	No regulations on storm water disposal into surface water.
New Jersey DOT		X				Retention basins only.
New York DOT	X		Basins and wells	Laboratory-Specific surface method. Field-Falling head test	6 months	
North Carolina DOT		X	Limited use[2] of vertical wells		Periodic inspection and clean out	State Health Dept. prohibits direct discharge of storm water into underground acquifers.

Ohio DOT	X		Open sumps[3] and dry wells	Grain size analysis		Legal restrictions prevent groundwater pollution.
Oregon State Hwy. Div.	X		Limited use of dry wells	Laboratory falling head test	Annual	No statues to prevent use of recharge systems, but such systems require a review of water quality.
Pennsylvania DOT	X		Vertical[1] wells and puts filled with 4-inch (102 mm) diameter stone		Periodic inspection	Restrictions on practice of groundwater recharge by storm water.
Rhode Island DOT		X				
South Dakota DOT		X		AASHTO T215-70 Modified		
South Carolina Hwy Dept.		X				
Tennessee DOT		X				
Texas Dept. of Hwys. and Public Trans.		X				
Utah DOT		X		Standard percolation test	6 months inspection 12 months clean out	Infiltration systems require approval of State Div. of Health and Water Rights
Vermont Div. of Hwys.		X				
Virginia Dept. of Hwys and Transportation		X				Infiltration systems used by local agencies only.

Wisconsin DOT		X				Use of deicing salts makes infiltration drainage unattractive
Wyoming State Hwy Dept.		X		Laboratory-Grain size analysis. Field-Concentric rings, standards percolation and pump tests.		Retention ponds only.Infiltration not considered in design. Serious groundwater contamination could occur in residential areas where private wells exist.
Highway Authority, Commonwealth of Puerto Rico		X				
Butte County, Calif.		X				
Colusa County, Calif.		X				
Dade County, Calif.	X		Basins and trenches	Static and falling head permeability tests in field.	Inspected annually and cleaned as required	
Fresno County, Calif.	X		Basins and wells			
Kern County, Calif.	X		Basins and riverbeds	Test pits	Annual inspection with infrequent clean out	No procedure for determining quantity of runoff
Leon County, Florida		X			Varies with site conditions	Leon County Ordinance 73-10 controls runoff.
Monteray County, Calif.	X		Percolation, detention ponds and trenches	Grain size analysis	Annual inspection and clean out	No restrictions on recharge.

Marin County, Calif.		X				
Napa County, Calif.	X					Experimental only.
Orange County, Environmental Agency, Santa Ana, Calif.	X		Basins and wells	Experience		Primary restriction is economics.
Orange County Florida	X		Basins and trenches	Grain size analysis and percolation test		
Riverside County, Calif.	X		Basins	Visual	Annual	Groundwater basin and downstream water rights prevent diversion of surface water.
Sacramento County, Calif.		X				State Health Dept. requirements place restrictions on infiltration systems.
San Joaquin County, Calif.		X			Annual inspection with clean out 1 to 5 years	
San Luis Obispo County, Calif.	X		Basins	Standard Percolation test		
Santa Barbara County, Calif.	X		Basins and vertical wells	Soil maps and experience		
Santa Clara County, Calif.	X		Basins, ponds, natural streams and vertical wells	Visual inspections, soil maps and standard percolation tests	Annual	
Solano County, Calif.		X				
Stanilaus County, Calif.	X		Vertical dry wells	Visual inspection	Inspection and clean out annually	Environmental requirements limit depth to 40 feet

Ventura County, Calif.		X				Natural recharge only
Yolo County, Calif.		X				Concerned with contamination of groundwater.
Yuba County, Calif.		X			Inspected 2 to 3 times/ yr. clean out once a year	Health Dept. is concerned with contamination of groundwater.
Boulder, Colorado	X		Pits and detention ponds	Not measured	Visual inspection and clean out when facility becomes half full	
Fresno, Calif.	X		Basins			
Lodi, Calif.		X			Annual	Detention basins
San Jose, Calif.		X				
Riverside, Calif.		X			Visual inspection and clean out as required	
San Diego, Calif.		X		SCS soil classification maps	Visual inspection and cleaning twice a year	
Contra Costa County Flood Control and Conservation Dist., Calif.	X		Basins	Single ring		High clay content soils percent general usage.
Fresno Metropolitan Flood Control Dist., Calif.	X		Basins	Laboratory-Constant head and falling head tests	Discing twice a year and mowed weekly during growing season	Groundwater at great enough depth that natural filtration or storm water offers.
Honolulu Board of Water Supply		X				State Health regulation control infiltration system for storm water.

Kings River Conservation Dist. Fresno, Calif.		X				
Los Angeles County Flood Control Dist.	X		Basins, pits and streambeds	Soil permeability determined by analyses of well lithologic logs and electric logs	Depends on storm frequency and magnitude plus duration of storm season	Bottoms of basins are ripped or disc when percolation rates decline. Percolated water provides 35 to 45 percent of the domestic water supply Southern California.
Metropolitan Water District of Southern Calif.		X				
Muskegon County Waste Water Management System, Michigan		X		Piezometer method by Kirkham		
Ottowa County Health Dept. Grand Heron, Mich.		X				
San Bernadino County Flood Control Dist., Calif.	X		Basins, pits, and sumps	Test pits	Inspected quarterly and cleaned once a year	
Water Quality Association Lombard, Illinois		X				
Wisconsin Dept. of Natural Resources		X				
Andrews and Clark Consulting Engrs, N.Y.	X		Basins and wells	Cased borings, test pits and laboratory tests		

Donahue and Assoc., Inc., Consultant, Wisc.		X				
Foth and Van Dyke and Assoc.,Inc., Consultant Green Bay, Wisc.		X				Concerned with wastewater only.
John A. Grant Jr. Consulting Engr., Boca Raton, Florida	X		Trench and lake retention	Standard percolation test		
Harza Engineering Co., Chicago		X	Recharge wells and pits	Slug test with shallow holes		Plan to use in Chile, S.A.
1 Limited use only						
2 Past experience only						
3 Permits requires on future systems.						

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Field Performance of Existing Infiltration Trench Systems

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Dade County Florida (1977)

Type of System	Year Constructed	Trench Size (ft.) Length x Width x Depth	Location
Slab Covered Trench with outfall pipe	1962	21,000 x 3 x 7	S.W. 42 Avenue
Slab Covered Trench	1963	5,000 x 5 x 7	Coral Way
Slab Covered Trench with out fall pipe	1965	5,000 x 3 x 8	N.W. 12 Avenue
Slab Covered Trench with outfall pipe	1965	5,000 x 3 x 8	N.W. 17 Avenue
Slab Covered Trench with outfall pipe	1965	13,000 x 4 x 7	N.W. 22 Avenue
Slab Covered Trench	1965	18,000 x 3 x 7	S.W. 37 Avenue
Slab Covered Trench with outfall pipe	1967	2,000 x 3 x 7	West 17 Avenue
Slab Covered Trench with outfall pipe	1967	11,000 x 3 x 7	Coral Way
Slab Covered Trench with outfall pipe	1967	26,000 x 3 x 7	S.W. 72 Street
Slab Covered Trench with outfall pipe	1976	16,000 x 5 x 7	Coral
Slab Covered Trench with outfall pipe	1976	1,600 x 3 x 7	S.W. 87 Avenue
Slab Covered Trench with outfall pipe	1976	11,000 x 3 x 7	S.W. 288 Street
Slab Covered Trench	1976	8,000 x 3 x	S.W. 312 Street
Slab Covered Trench/ French Drain	1975	2,000 x 3 x 7 and 5 x 7 with fully perforated 36 in. diameter pipe	N.E. 151 Street
Slab Covered Trench/ French Drain with outfall pipe	1976	11,000 x 3 x 7 and 5 x 7 with fully perforated 36 in. diameter pipe	N.W. 22 Avenue
Notes: 1 inch = 25.4 cm, 1 foot = 0.305 m			
The listed facilities have functioned properly, with minimum maintenance performed since installation. Each installation is considered a success in providing an economical and environmentally compatible system.			

Toronto, Ontario, Canada			
Type System	Year Constructed	Description	Location

Perforated Detention Tank with outfall pipe	1978	240 ft of fully perforated 96-inch diameter pipe (detention tank) with aggregate backfill, flow regulator and outfall pipe	Heart Lake subdivision, Toronto, Ontario Canada

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Performance Test Program, Dade County Florida

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1. Effect of Soil Migration on Performance of Infiltration Trench Systems

Laboratory model tests were conducted by the Dade County Department of Public Works in Miami, Florida using a rectangular wooden box with plexiglass sides in order to visually study the action of water passing through the interface of the natural soil and coarse rock backfill. The box was 6 feet (.83 m) in length, 3 feet (0.92 m) in width, and 3 1/2 feet (1.07 m) in height. The plexiglass extended about 2/3 of the way up the sides. Two 4 inch (101.6 mm) diameter PVC pipes were placed vertically in the box approximately 2 feet (0.61 m) from each end. These pipes had forty 3/8 inch diameter (9.5 mm) diameter holes drilled uniformly in the bottom 2 feet (0.61 m) as shown in Figure C-1. The box was filled with sand very carefully with a hand shovel, allowing the sand to fall on its natural angle of repose for half the length of the box. No external compaction effort was used, in order to allow maximum infiltration of sand. There was a 3 inch (76.2 mm) diameter hole placed at the bottom of each end to allow water to drain out of the box. Filter X plastic cloth material was placed on the inside of the box over the holes in order that the sand and coarse rock backfill would not spill out. Next, coarse rock backfill was carefully placed in the other end of the box until it completely covered the sand. A 12 inch (0.30 m) wide strip of filter cloth (Filter X) was placed between the sand and the coarse rock on one side of the box to study its effect on soil infiltration. Tap water from a 3/4 inch (19.1 mm) hose was discharged into the vertical perforated 4 inch (101.6 mm) PVC pipe at the coarse rock end of the box and the water was allowed to rise to the top of the box. The hose was removed and the water was allowed to drain out of the ends of the box. This took approximately 1/2 hour. The procedure was repeated two more times. This simulated the movement of storm water through a trench-type drainage system out to the adjacent soil.

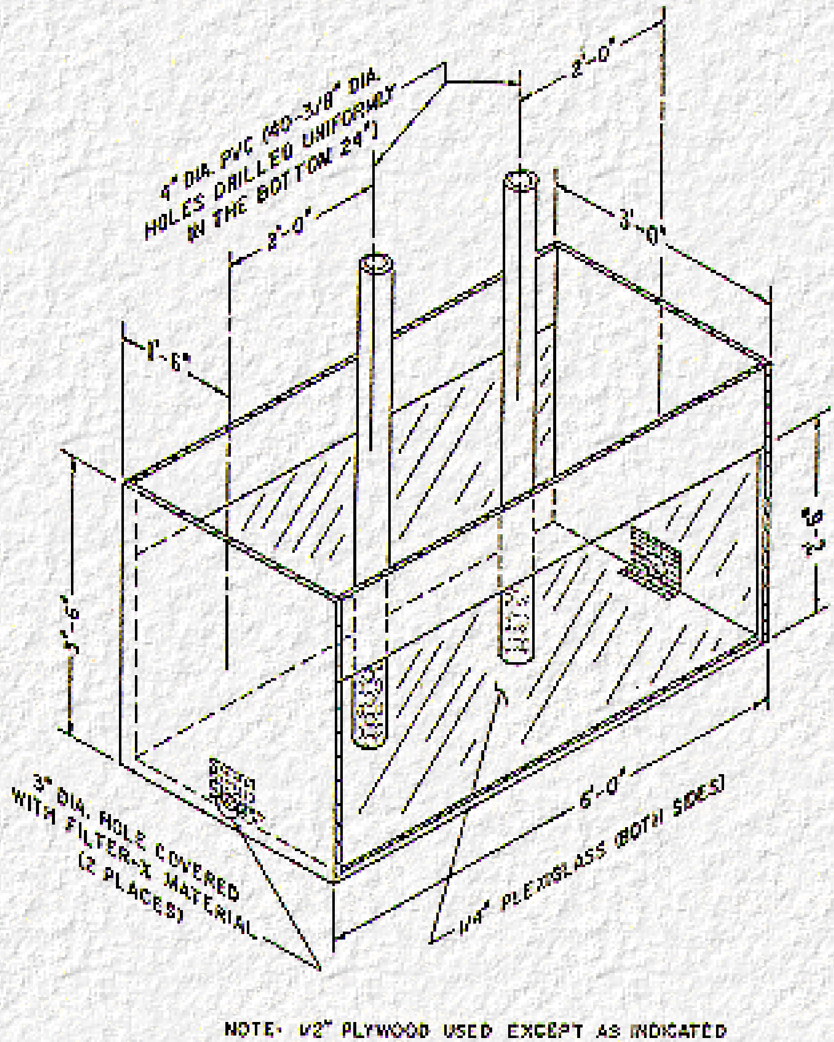

Figure C-1. Laboratory Model to Visually Show the Effect of Soil Infiltration onto the Course Aggregate Dispersing Medium. *(Courtesy of Dade County, Dept. of Public Works, Miami, Florida)*

Visual inspection was made through the plexiglass to see how much sand infiltrated into the coarse rock backfill. It appeared that the amount of lateral infiltration or contamination was less than one inch (25 mm) into the backfill.

During the second phase of testing, water was discharged into the vertical pipe at the end of the box with the sand in it. This was done to simulate a condition where water is passing from the natural soil into the coarse rock backfill.

The box was left outdoors and the test was repeated about a month and a half after the initial test. It appeared that no appreciable amount of sand infiltrated into the coarse rock. It is believed that this was due to the fact that the sand-coarse rock interface was in equilibrium. Several months later the entire test was repeated. This time a wooden board was installed vertically to separate the coarse rock from the sand. As the box was filled the board was removed, leaving a vertical plane between the sand and the coarse rock. Again no external compaction effort was used during loose placement of the material.

Water was introduced through the vertical pipe into the coarse rock. As the water rose in the box there was visual evidence of voids appearing at the coarse rock-sand interface. As the void size increased, settlement occurred on the top surface of the sand. The water was allowed to drain out of the box and the test was repeated two more times. These tests indicated very little evidence of additional void development between the coarse rock backfill and sand interface and no subsequent additional settlement on the top surface of the sand. This suggests that no further appreciable settlement will take place due to compaction of the sand by the submerging action of the water, once it becomes saturated and then dewatered.

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2. Effect of Hole Size, Number and Distribution on Discharge From Perforated Pipe With Aggregate Backfill

A 15 inch (381 mm) diameter uniformly perforated metal pipe, 3 feet (0.92 m) in length with welded bottom was inserted 3 feet (0.92 m) vertically in the top of an aggregate pile approximately 12 feet (3.7 m) high and 30 feet (9.2 m) in diameter. A water supply truck and a pump with a 3 inch (76 mm) diameter hose was used to discharge water into the top of the pipe. The depth of a constant water head in the pipe was then recorded. The results give a good idea of what the discharge rate is through the perforations and the voids of coarse aggregate backfill in a condition unrestricted by natural surrounding soil.

Pipe with 3/16 inch (4.8 mm) or 3/8 inch (9.5 mm) diameter perforations ranging from 47 to 229 perforations per lineal foot was installed and tested in coarse aggregate (3/4 inch x 1 1/2 inch) (19 mm x 38 mm), medium aggregate (1/4 inch x 3/8 inch) (6.4 mm x 9.5 mm), and fine aggregate sand No. 50 x No. 4, all having between 40 and 50% voids. A 55 gallon (0.22 m^3) drum was then filled with water and that time was recorded to determine discharge rate of pump for each test. Table C-1 presents a brief summary of the test results.

After reviewing the results of the above tests it was determined that additional tests should be run with a sample having 150 holes per lineal foot, since it was felt that this would be an optimum number of holes from a manufacturing and engineering standpoint. These test results are shown in Table C-2.

Table C-1. Summary of Tests Made in 1974 Typifying Unrestricted Flow Through Aggregate Backfill ***(Dade County DPW, Miami, Fla.)***

Pump-Discharge (Inflow - cfs)	Height of Constant Water Head (Inches)	Course Aggregate Size (Inches)	Number of 3/8-inch Diam. Perforations per Lineal Foot of Pipe	Discharge though Pipe and Aggregate (cfs)
0.271	14	3/4 x 1 1/2	182 (46.3/ft^2)	0.232
"	13	"	196 (49.9/ft^2)	0.248
"	12	"	222 (56.5/ft^2)	0.271

"	12	"	229 (58.3/ft^2)	0.271
0.293	24	1/4 x 3/8	182 (46.3/ft^2)	0.147
"	22	"	196 (49.9/ft^2)	0.160
"	20	"	220 (56.5/ft^2)	0.176
"	18	"	229 (58.3/ft^2)	0.195
0.301	36*	"	47 (12.0/f^2)	0.100*
"	22**	3/4 x 1 1/2	47 (12.0/ft^2)	0.172
Note : 1 inch = 25.4 mm = 2.54 cm				
*Pipe overflowed				
**Filled pipe with coarse aggregate to 21 Inch depth. Water reached a 22 1neh constant water head in less time. This indicated that storage was affected, but infiltration reduction was negligible.				

Table C-2. Summary of Tests Made in 1975 Typifying Restricted and Unrestricted Flow through Aggregate Backfill.

Pump-Discharge (Inflow - cfs)	Height of Constant Water Head (Inches)	Course Aggregate Size (Inches)	Number of 3/8-inch Diam. Perforations per Lineal Foot of Pipe	Discharge though Pipe and Aggregate (cfs)
0.207	16	1/4 x 3/8	150-3/8 (38.17/ft^2	---
0.207	21.5	"	"	Filter X fabric wrapped around outside of pipe
0.207	28.5	"	150-3/16 (1.05 in.^{2} /ft^2)	---
0.207	36	Sand (no filter cloth)	"	Water overflowed top at 36 inch height in 30 seconds. Water drained out of pipe to a point 4 inches above bottom in 60 seconds. Last 4 inches drained very slowly.

0.210	36	Sand (filter cloth)	150-3/8 (38.17/ft^2)	Filter X fabric wrapped around outside of pipe. Water overflowed 36 inch height in 40 seconds. Water drained from 36 inches to 13 inch depth in 60 seconds and from 13 inch to-7 inch-depth in 60 seconds. The last 4 inch drained very slowly.
0.210	36	Sand (No filter cloth)	150-3/8 (38.17/ft^2)	Water overflowed 36 inch height in 30 seconds. Sand came into pipe through 3/8 inch diameter holes. Water drained from 36 inch to 18 inch height in 60 seconds. The remaining height drained very slowly. Sand infiltration could be observed.
0.210	10	3/4 x 1 1/2 (filter cloth)	150-3/8 (38.17/ft^2)	Filter X fabric wrapped around out side of pipe. Ten inch height at start of test. Three shovels of sand added to inside of pipe and stirred with water hose. Water height increased to 14 inch head. Sand completely washed out of pipe after 2 minutes and water level returned to 10 inch head. Conclusion ; water apparently passed down between filter cloth and outside of pipe.
Note: 1 inch = 25.4 mm = 2.54 cm, 1 cfs = 0.028m3/sec.				

It can be concluded from these tests (Table C-1 and Table C-2) that:

a.) Cohesionless fines will migrate into the coarse aggregate backfill unless a filter fabric or graded aggregate filter is provided at the interface between cohesionless-fine grained native soil and coarse aggregate pipe backfill.

b.) Coarse aggregate backfill is necessary to provide storage, prevent clogging, and prevent restriction of flow from pipe to native soil.

c.) The application of filter cloth around the pipe periphery tends to cause clogging of pipe perforations due to buildup of fines.

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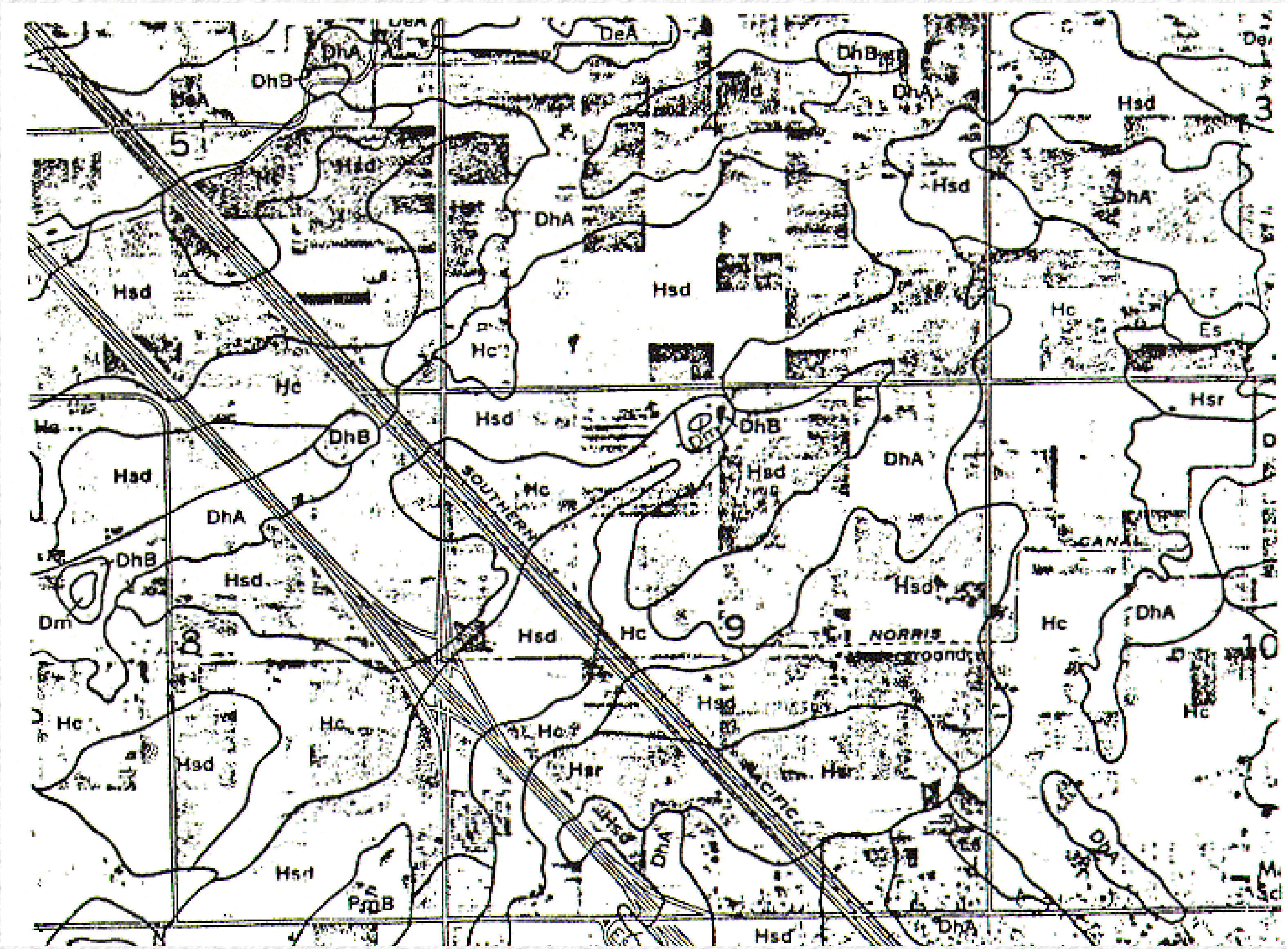
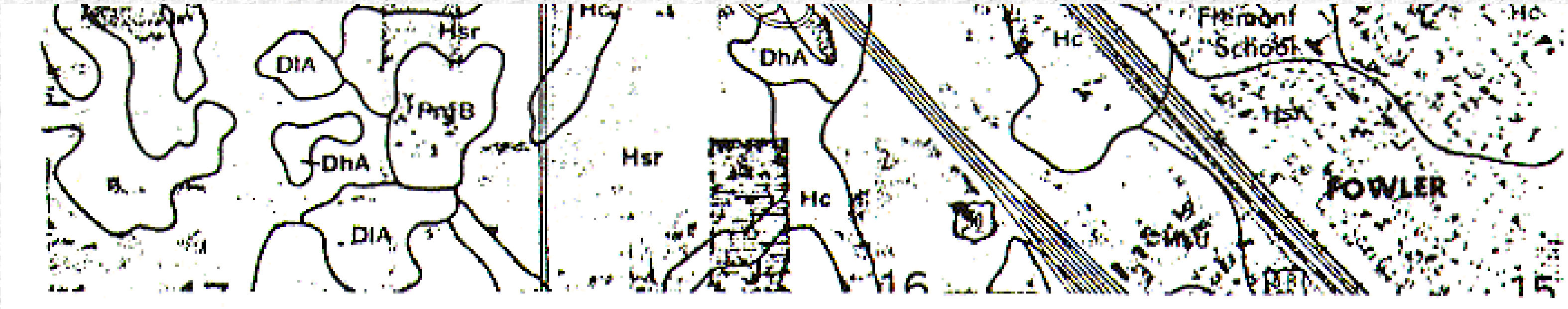
D.1 U.S. Department of Agriculture, Soil Conservation Service Soil Classification Maps

Maps giving soil types based on shallow test holes and reconnaissance of surface conditions are available for many areas of the United States. A typical map is given below. Refer to the "Soil Legend" on the following page for soil type.





Typical Soil Conservation Service Soil Classification Map

Soil Legend

Each symbol consists of letters of a combination of letters and numbers. The first letter is the initial one of the soil name. A second capital letter, if used, shows the class of slope. Soils for which no slope letter is shown are nearly level. A final number, 2, in a symbol shows that the soil is eroded.

Cm	Chino sandy loam, saline-alkali
Cn	Chino fine sandy loam
Co	Chino fine sandy loam, saline-alkali
Cp	Chino fine sandy loam, moderately deep, saline-alkali
Cr	Chino loam
Cs	Chino loam, saline-alkali
CtA	Chualar sandy loam, 0 to 3 percent slopes
CtB	Chualar sandy loam, 3 to 9 percent.
CuC	Cibo clay, 3 to 15 percent slope.
CuD	Cibo clay, 15 to 30 percent
CuE	Cibo clay, 30 to 45 percent
CvD	Cibo very rocky clay, 3 to 30 percent slopes
CvE	Cibo very rocky clay, 30 to 45 percent slopes
CvF	Cibo very rocky clay, 45 to 70 percent slopes
CwD	Cibo extremely rocky clay, 3 to 30 percent slope
CwE	Cibo extremely rocky clay, 30 to 45 percent slopes
CxC	Coursegold fine sandy loam, 9 to 15 percent slopes
CxF	Coursegold fine sandy loam, 15 to 30 percent slopes
CxE	Coursegold fine sandy loam, 30 to 45 percent slopes

CxF	Coursegold fine sandy loam, 45 to70 percent slopes
CyF	Coursegold fine sandy loam, 45 to 70 percent slopes
CzF	Colluvial land
CzaB	Cometa sandy loam, 3 to 9 percent slopes
CzaC	Cometa sandy loam, 9 to 15 percent slopes
CzaD	Cometa sandy loam, 15 to 30 percent slopes
CzbB	Cometa loam, 2 to 9 percent slopes
CzcB	Cometa-San Joaquin sandy loams, 3 to 9 percent
DeA	Delhi sand, 0 to 3 percent slopes
DeB	Delhi sand, 3 to 9 percent slopes
DhA	Delhi loamy sand, 0 to 3 percent slopes
DhB	Delhi loamy sand, 3 to 9 percent slopes
DIA	Delhi loamy sand, moderately deep, 0 to 3 percent slopes
Dm	Dello loamy sand
Dn	Dello sandy loam
DpE	Delpiedra extremely stony loam, 30 to 45 percent slopes
DpF	Delpiedra extremely stony loam, 45 to 70 percent slopes
DsF	Delpiedra-Fancher extremely stony loams, 45 to 70 percent slopes
Ec	El Peco sandy loam
Ed	El Peco fine sandy loam
Ep	El Peco loam
Es	Exeter sandy loam
Et	Exeter sandy loam, shallow
Ex	Exeter loam
FaB	Fallbrook sandy loam, 3 to 9 percent slopes
FaC	Fallbrook sandy loam, 9 to 15 percent slopes
FaD	Fallbrook sandy loam, 15 to 30 percent slopes

FaE	Fallbrook sandy loam, 30 to 45 percent slopes
FbB	Fallbrook sandy loam, shallow, 3 to 9 percent slopes
FbD	Fallbrook sandy loam, shallow, 9 to 30 percent slopes
FcD	Fallbrook very rocky sandy loam, 3 to 30 percent slopes
FcF	Fallbrook very rocky sandy loam, 30 to 70 percent slopes
FdD	Fallbrook very rocky sandy loam, shallow, 3 to 30 percent slopes
FdF	Fallbrook very rocky sandy loam, shallow, 30 to 70 percent slopes
FeE	Fallbrook extremely rocky sandy loam, shallow, 30 to 45 percent slopes
FhE	Fancher extremely stony loam, 30 to 45 percent slopes
FhF	Fancher extremely stony loam, 45 to 70 percent slopes
FlE	Fancher-Blasingame complex, 30 to 45 percent
FlF	Fancher-Blasingame complex, 45 to 70 percent
Fm	Foster sandy loam
Fn	Foster loam
Fo	Foster loam, saline-alkali
Fp	Foster loam, moderately deep
Fr	Foster loam, moderately deep, saline-alkali
Fs	Fresno sandy loam
Ft	Fresno sandy loam, shallow
Fu	Fresno fine sandy loam
Fv	Fresno fine sandy loam, shallow
Fw	Fresno clay loam
Fx	Fresno-Traver complex
FyD	Friant fine sandy loam, 9 to 30 percent slopes
FyE	Friant fine sandy loam, 30 to 45 percent slopes
Ga	Grangeville sandy loam
Gd	Grangeville sandy loam, saline-alkali
Ge	Grangeville sandy loam, sandy substratum
Gf	Grangeville fine sandy loam
Gg	Grangeville fine sandy loam, saline-alkali

Gh	Grangeville fine sandy loam, water table,
Gk	Grangeville fine sandy loam, water table, saline-alkali
Gl	Grangeville fine sandy loam, gravelly substratum
Gm	Grangeville fine sandy loam, sandy substratum
Gn	Grangeville fine sandy loam, hard substratum
Go	Grangeville fine sandy loam, hard substratum, saline-alkali
Gp	Grangeville soils, channeled
GrF	Granite rock land
GsA	Greenfield course sandy loam, 0 to 3 percent slopes
GtA	Greenfield sandy loam, 0 to 3 percent slopes
GtB	Greenfield sandy loam, 3 to 9 percent slopes
GuA	Greenfield sandy loam, moderately deep, 0 to 3 percent slopes

Ha	Hanford coarse sandy loam
Hb	Hanford coarse sandy loam, hard substratum
Hc	Hanford sandy loam
Hd	Hanford sandy loam, benches
He	Hanford sandy loam, gravelly substratum
Hf	Hanford sandy loam, sand substratum
Hg	Hanford sandy loam, silty substratum
Hh	Hanford sandy loam, clay loam substratum
Hk	Hanford sandy loam, hard stratum
Hl	Hanford gravelly sandy loam
Hm	Hanford fine sandy loam
Hn	Hanford fine sandy loam, gravelly substratum
Ho	Hanford fine sandy loam, silty substratum
Hp	Hanford fine sandy loam, clay loam substratum
Hr	Hanford fine sandy loam, hard substratum
Hsa	Hesperia course sandy loam
Hsc	Hesperia course sandy loam, saline-alkali

Hsd	Hesperia sandy loam
Hse	Hesperia sandy loam, saline-alkali
Hsm	Hesperia sandy loam, moderately deep
Hsn	Hesperia sandy loam, moderately deep, , saline-alkali
Hso	Hesperia sandy loam, shallow
Hsp	Hesperia sandy loam, shallow, saline-alkali
Hsr	Hesperia fine sandy loam
Hss	Hesperia fine sandy loam, saline-alkali
Hst	Hesperia fine sandy loam, moderately deep
Hsy	Hesperia fine sandy loam, moderately deep, saline-alkali
HtC	Hideaway extremely stony loam, 3 to 15 percent slopes
Hu	Hildreth Clay
HvE	Holland coarse sandy loam, 15 to 45 percent slopes
HwA	Honcut, fine sandy loam, 0 to 3 percent slopes
HwB	Honcut, fine sandy loam, 3 to 9 percent slopes
HyA	Honcut fine sandy loam, gravelly substratum, 0 to 3 percent slopes
HzA	Honcut fine sandy loam, hard substratum, 0 to 3 percent slopes
KeC	Keefers loam, 3 to 15 percent slopes
KfD	Keefers cobbly loam, 3 to 30 percent slopes
KmC	Keyes cobbly clay loam, 3 to 15 percent slopes
LbB	Los Robles sandy loam, 2 to 9 percent
LgB	Los Robles sandy loam, gravelly substratum, 2 to 9 percent
LmA	Los Robles loam, 0 to 3 percent slopes
LmB	Los Robles loam, 3 to 9 percent slopes
LnB	Los Robles loam, hard substratum, 2 to 9 percent
LoA	Los Robles clay loam, 0 to 3 percent
Ma	Madera sandy loam

Mc	Madera loam
Md	Madera loam, saline-alkali
Me	Madera clay loam
Mf	Merced clay loam
Mg	Merced clay loam, slightly saline
Mh	Merced clay,
Mk	Merced clay, slightly saline
Ml	Merced clay, moderately saline
Mm	Merced clay, saline-alkali

D.2 Specific Surface Method of New York State DOT

The following pages of this appendix give selected portions of the New York State Department of Transportation, Soil Mechanics Bureau, Albany, New York, "Test Procedure for Specific Surface Analysis", STP-1, Aug. 27, 1973, with its Appendix A - APPLICATION OF RESULTS, and its Appendix E, SAMPLE PROBLEM. Portions of that report giving its THEORETICAL BACKGROUND, a FLOW CHART FOR COMPUTER SOLUTION, and a COMPUTER PROGRAM LISTING are not included in this Appendix in the interest of reducing the volume of this manual. Those wanting to study these subjects are referred to the complete New York report.

1. Scope

This test procedure describes the method of determining the specific surface of soil solids from grain size distribution data. Also described is the use of the specific surface in calculating the coefficient of permeability and other soil properties.

Appendices contain background information regarding this test method and an example problem.

2. Definitions

A. Specific Surface (S) - The particle surface area contained in a unit volume of soil solids. The particle surface area includes only the external particle surface (the internal porosity of individual particles is neglected).

B. Spherical Specific Surface (S_s) - In the analysis of grain size distribution date all grains are initially assumed to be of spherical shape, yielding specific surface on that basis. This result is later corrected to account for the actual grain shape. For an individual grain or aggregation of grains of uniform spherical diameter (d),

$$S_s = \frac{\text{Particle Surface Area}}{\text{Particle Volume}} = \frac{d^2}{d^3/6} = \frac{6}{d}$$

C. Shape Factor (f) - A factor applied to Ss to convert it to the actual specific surface, S. The factor is obtained by comparison of actual grain shapes with standard reference shapes and factors, where

$$1.0 \leq f \leq 1.8 \text{ and}$$
$$S = f \bullet S_s$$

3. Summary of Method

Using the method described in this manual, the specific surface is determined from an analysis of the grain-size distribution of the soil and the shape of The individual grains. The procedure consists of the following main steps:

1) A grain-size analysis of a soil specimen.

2) An examination of the shape characteristics of the grains contained in each sieve interval.

3) An arithmetic or computer calculation of the specific surface based on the data obtained in steps 1 and 2.

4. Equipment

The equipment required for this test is the same as that required for STM-2, Test Method for the Grain-Size Analysis of Granular Soil Materials, with the following modifications and additions:

1) The series of sieves used for the grain-size analysis should preferably be such that:

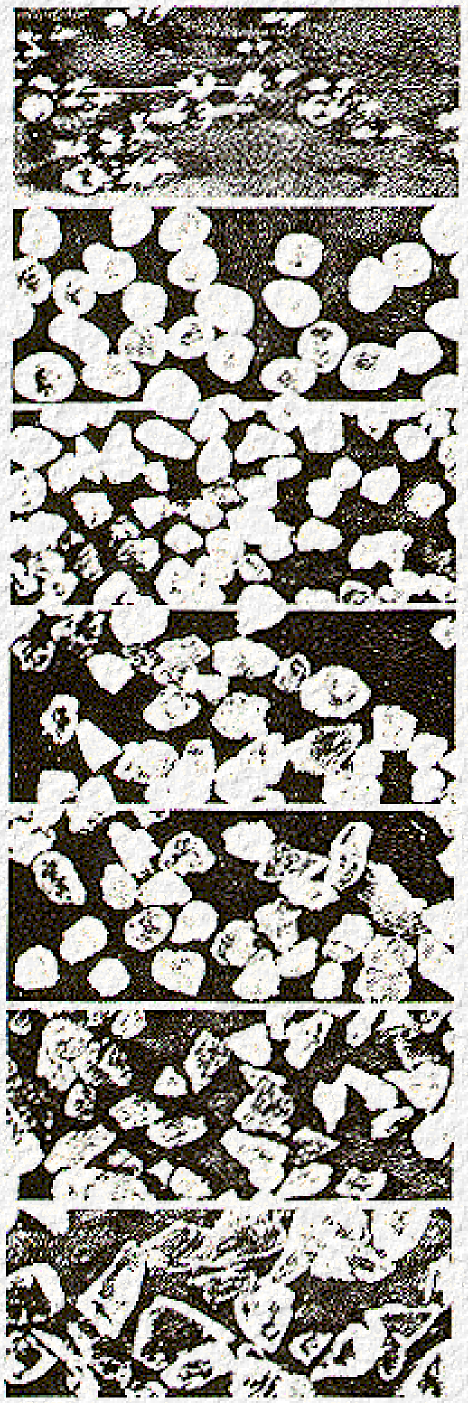
$$\frac{d_n}{d_{n+1}} = \frac{d_{n-1}}{d_n}$$

where the letter d denotes the size of the sieve opening, and the subscript n refers to the numerical position of the sieve in the series. The sieves recommended for a specific surface analysis are designated in Table 1.

2) Small containers for holding 5 to 10 grams of material for shape factor determination.

Table 1. U.S. Sieve Series Data (Selected Sizes)

Size Designation	Openings in cm (d)	Specific Surface of Sphere with Diameter=d in cm^2/cm^3
4"	10.16	0.5906
*3"	7.61	0.7884
2"	5.08	1.1811
*1½"	3.81	1.5748
1"	2.54	2.3622
*¾"	1.91	3.1414
½"	1.27	4.7244
*3/8"	0.951	6.3091
¼"	0.635	9.4488
*#4	0.476	12.6050
*#8	0.238	25.2101
#10	0.200	30.0000
*#16	0.119	50.4202
#20	0.084	71.4286
*#30	0.0595	100.8406
#40	0.0420	142.8571
*#50	0.0297	202.0202
#60	0.0250	240.0000
*#100	0.0149	402.6846
*#200	0.0074	810.8108
#400	0.0037	1621.6216
*Recommended sizes for specific surface data		



f=1.0 Glass beads.

f=1.15 Ottawa calibration sand.

f=1.25 Dredged sand from vic. of Staten Island.

f=1.35 Cow Bay sand.

f=1.45 2Q sand (utilized in field density tests).

f=1.6 Crushed limestone.

f=1.8 Crushed Pyrex glass.

Figure 1. Particle Angularity Factors

3) A magnifying glass or low power microscope examining the shape characteristics of the individual soil particles in the finer sizes.

4) A series of photos showing particles of various shapes and the corresponding shape factors. This series of photos is reproduced in Figure 1 of this manual.

5. Procedure

5.1 Sample and Specimen Sizes

For this procedure, a sample is defined as a representative portion of the material obtained in the field for the purpose of testing. A specimen is that portion of the sample actually subjected to testing. The desirable weight of soil samples and specimens depends on the maximum particle size. The following can be used as a guide:

Maximum Particle Size	Minimum Sample Weight	Recommended Specimen Weight
4 in.	40 lbs.	25 lbs.
3 in.	40 lbs.	15 lbs.
2 in.	40 lbs.	4000 gms. or 10 lbs.
1 1/2 in.	40 lbs.	3200 gms. or 7.5 lbs
1 in.	40 lbs.	2500 gms. or 5 lbs.
3/4 in.	15 lbs.	1,200 gms.
3/8 in.	7.5 lbs.	600 gms.
1/4 in. or smaller	5 lbs.	400 gms.

5.2 Grain-size Analysis

The grain-size analysis is performed basically as described in STM-2, Test Method for the Grain Size Analysis of Granular Soil Materials. However, instead of being separated into plus and minus 1/4-inch fractions, the soil is separated into plus and minus 3/8-inch fractions. The grain-size analysis data is recorded on Form SM 389 as follows:

SM 389a (9/73) NEW YORK STATE - DEPARTMENT OF TRANSPORTATION

SPECIFIC SURFACE OF GRANULAR MATERIALS

P.I.N. ______
PROJECT ______

DRILL HOLE NO.	SAMPLE NO.	DEPTH

REGION ____ COUNTY ______

TESTED BY ______ REGION/MAIN OFFICE ______ DATE ______

SAMPLE INFORMATION			
A	Wt. + 3/8" Dry	Lbs.	
B	Wt. - 3/8" Moist	Lbs.	
C	Wt. - 3/8" Dry (B ÷ (1+R))	Lbs.	
D	Wt. Total Specimen Dry (A+C)	Lbs.	
E	Fraction Ret. on 3/8" (A÷D)		
F	Fraction Passing 3/8" (1-E)		
G	Dry Wt. - 3/8" Before Wash	Gms	
H	Dry Wt. - 3/8" After Wash	Gms	
I	Wt. Mat'l Lost in Wash (G-H)	Gms	

MOISTURE CONTENT			
J	Tare No.		
K	Wt. Tare & Wet Soil	Gms	
L	Wt. Tare & Dry Soil	Gms	
M	Wt. Tare	Gms	
N	Wt. of Water (K-L)	Gms	
P	Wt. Dry Soil (L-M)	Gms	
R	% Moisture ((N÷P) x 100)		%

1 Sieve Des.	2 Opening Size (CM)	3 W.T. Retained	4 Fraction Ret. (Partial) NOTE 2	5 Fraction Ret. (Total) NOTE 3	6 Shape Factor	7 d_2 (CM) NOTE 4	8 $\frac{d_1}{d_2}$ NOTE 4	9 K (From Fig.-2)	10 $(S_a)_{av}$ (CM^2) 9÷7	11 SPECIFIC SURFACE Spherical (CM^2/CM^3) 5 x 10	12 SPECIFIC SURFACE Corrected (CM^2/CM^3) 11 x 6
3"	7.61					7.61	2.0	8.7	1.14		
1-1/2"	3.81					3.81	2.0	8.7	2.28		
3/4"	1.91					1.91	2.0	8.7	4.55		
3/8"	0.951					0.951	2.0	8.7	9.15		
#4	0.476					0.476	2.0	8.7	18.3		
#8	0.238					0.238	2.0	8.7	36.6		
#16	0.119					0.119	2.0	8.7	73.1		
#30	0.0595					0.0595	2.0	8.7	146		
#50	0.0297					0.0297	2.0	8.7	293		
#100	0.0149					0.0149	2.0	8.7	584		
#200	0.0074					0.0074					
		Pan (plus Mat'l. lost in wash)									

CHECK TOTAL (=1.000) ______ TOTAL SPECIFIC SURFACE ______

NOTES

1. See manual for method of obtaining grain-size distr. for mat. passing the #200 sieve.
2. For plus 3/8-in. specimen, 4 = 3 ÷ A
   For minus 3/8-in. specimen, 4 = 3 ÷ G
3. For plus 3/8-in. specimen, 5 = 4 x E
   For minus 3/8-in. specimen, 5 = 4 x F
4. d_1 = opening size of upper sieve in a sieve interval.
   d_2 = opening size of lower sieve in a sieve interval.

LINE A: The air-dry weight of that portion of the specimen which is retained on the 3/8" sieve.

LINE B: The moist weight, without drying, of that portion of the specimen which passes through the 3/8" sieve. After this portion is weighed and the weight is recorded, a cruller specimen is taken from it for a moisture content determination (Lines J through R).

LINE C: The computed dry weight of the portion passing the 3/8" sieve. The dry weight is found by dividing the moist weight (Line B) by one plus the moisture content in decimal form (from Line R.).

LINE D: The total dry weight of the specimen, obtained by adding Lines A and C.

LINE E: The decimal fraction of the specimen retained on the 3/8" sieve (Line A divided by Line D).

LINE F: The decimal fraction of the specimen passing the 3/8" sieve (one minus Line E).

LINE G.: The dry weight of the portion of the minus 3/8" material to be used for a grain-size analysis.

LINE H: The dry weight of the material from Line G after washing it on a No. 200 sieve.

LINE I: The weight of the material lost through the No. 200 sieve (Line G minus Line H.).

LINE J: The number of the tare used for the moisture content determination.

LINE K: The weight of the tare and the wet soil.

LINE L: The weight of the tare and the soil after drying.

LINE M: The weight of the tare.

LINE N: The weight of the water (Line K minus Line L).

LINE P: The weight of the dry soil (Line L minus Line M).

LINE R: The moisture content of the soil in percent (Line N divided by Line P and multiplied by 100).

COLUMN 1: The designations of the sieves used in the specific surface analysis have been listed in this column.

COLUMN 2: The sizes (in centimeters) of the openings of the sieves used in the analysis have been listed in the upper 11 spaces in this column. The lower 4 spaces have been reserved for particle diameters obtained from a hydrometer analysis or by extrapolating the the results of the sieve analysis as explained subsequently in this section.

COLUMN 3: List, opposite each sieve, the weight of material retained on that sieve. Add the weight of the material in the pan (passing the No. 200 sieve) to the weight of the material lost-through the No. 200 sieve in washing and record the sum in the bottom space in this column.

COLUMN 4: For sieves with openings of 3/8" or greater, divide the weight retained on each sieve (Column 3) by the weight of the specimen retained on the 378" sieve (Line B). For sieves with openings smaller than 3/8", divide the weight retained on each sieve (Column 3) by the weight of the minus 3/8" specimen before washing (Line F.). Record in Column 4 to four places to the right of the decimal point (0.0000). For grain sizes antler than 0.0074 cm, record the fraction directly from a hydrometer analysis or as extrapolated from grain size distribution curve.

COLUMN 5: For sieves with openings of 3/8" or greater, multiply the values in Column 4 by the fraction of the total specimen retained on the 3/8" sieve (Line C). For particles smaller than 3/8", multiply the values in Column 4 by the fraction of the total specimen passing the 3/8" sieve (Line E). Record in Column 5 to four places to the right of the decimal point (0.0000).

If the material lost in the wash plus that passing the No. 200 sieve during sieving is not more than 5% of the total specimen, the grain-size distribution below the No 200 sieve may be estimated with little loss of accuracy by extrapolating the grain-size distribution curve to the horizontal axis of the grain-size distribution chart. The opening size at which the extrapolated curve intersects the horizontal (0% passing) axis is recorded in Column 2 as the lowest opening size. The material lost in washing and that passing through the No. 200 sieve during subsequent sieving is recorded as being retained on this lowest opening size. For non-plastic materials with more than 5% by weight passing the No. 200 sieve, a hydrometer analysis may be required in order to obtain a reliable grainsize distribution and specific surface. When obtained, the hydrometer results should be extrapolated in the same manner to estimate the 0% passing size.

If the material under investigation contains plastic fines, this procedure is not valid. Plasticity signifies the presence of clay minerals. The flat shape of the clay particles makes it very difficult, if not impossible, to use this type of analysis for the determination of specific surface. Furthermore, the physico-chemical activity of clay minerals would invalidate the methods presented herein for computing the permeability of a material with a known specific surface.

5.3 Determining Shape Factors

The shape factor for each grain-size interval is determined by comparing the shape of the grains in that interval against a series of photos (Figure 1) and recorded in COLUMN 6. These photos depict grains of standard shapes, and the corresponding shape factor is indicated beside each photo. Observed grain shapes frequently will lie between those in the standard photos, in which case the shape factor should be interpolated to best describe the observed shape. Also, the grains observed in a given interval may often have varying shapes. Shape variation usually follows size variation within the interval, so that the proportions can be visually estimated and a final weighted average shape factor can be recorded to satisfactorily represent the material contained in the interval.

As particle size diminishes, particle angularity (hence shape factor) characteristically increases. Commonly, particles in the range of 0.1 mm will have shape factors of 1.5 or more.

---

6. Calculations

6.1 General

The specific surface can be calculated arithmetically or by a computer, if available. Both methods are described herein.

6.2 Arithmetic Solution

Columns 7 through 12 on Form SM 385 are used to obtain the specific surface by arithmetic calculation. The horizontal lines in these columns are staggered with respect to those in Columns 1 through 6 because each value listed in Columns 7 through 12 refers to the material in an interval between two successive sieves listed in Column 1. The calculations required for each column are indicated briefly on Form SM 389 and explained in greater detail here:

COLUMN 7: d_1 is the upper limiting sieve opening size for a sieve interval and is obtained from Column 2.

COLUMN 8: The ratio d_1/d_2 to be recorded in this column is obtained simply by dividing d_1, defined above, by d_2, the lower limiting sieve opening size in the interval.

COLUMN 9: The value of K depends on the value of r $(=d_1/d_2)$ and can be obtained from Figure 2a for $r \leq 10$ and from Figure 2b for $10 \leq r \leq 100$.

COLUMN 10: The average spherical specific surface for a sieve interval, $(S_s)_{av}$ is obtained by dividing K (Column 9) by d_1 (Column

7). The result is recorded in Column 10.

COLUMN 11: The fractional spherical specific surface for an interval is obtained by multiplying $(S_s)_{av}$ from Column 10 by the decimal fraction of the total specimen in that interval. The product is recorded in column 11.

COLUMN 12: The specific surface is corrected for angularity of particles by multiplying each value in Column 11 by the shape factor for that interval. The results are recorded in Column 12. The sum of the values in this column is the total specific surface, in square centimeters per cubic centimeter of solids of the material investigated.

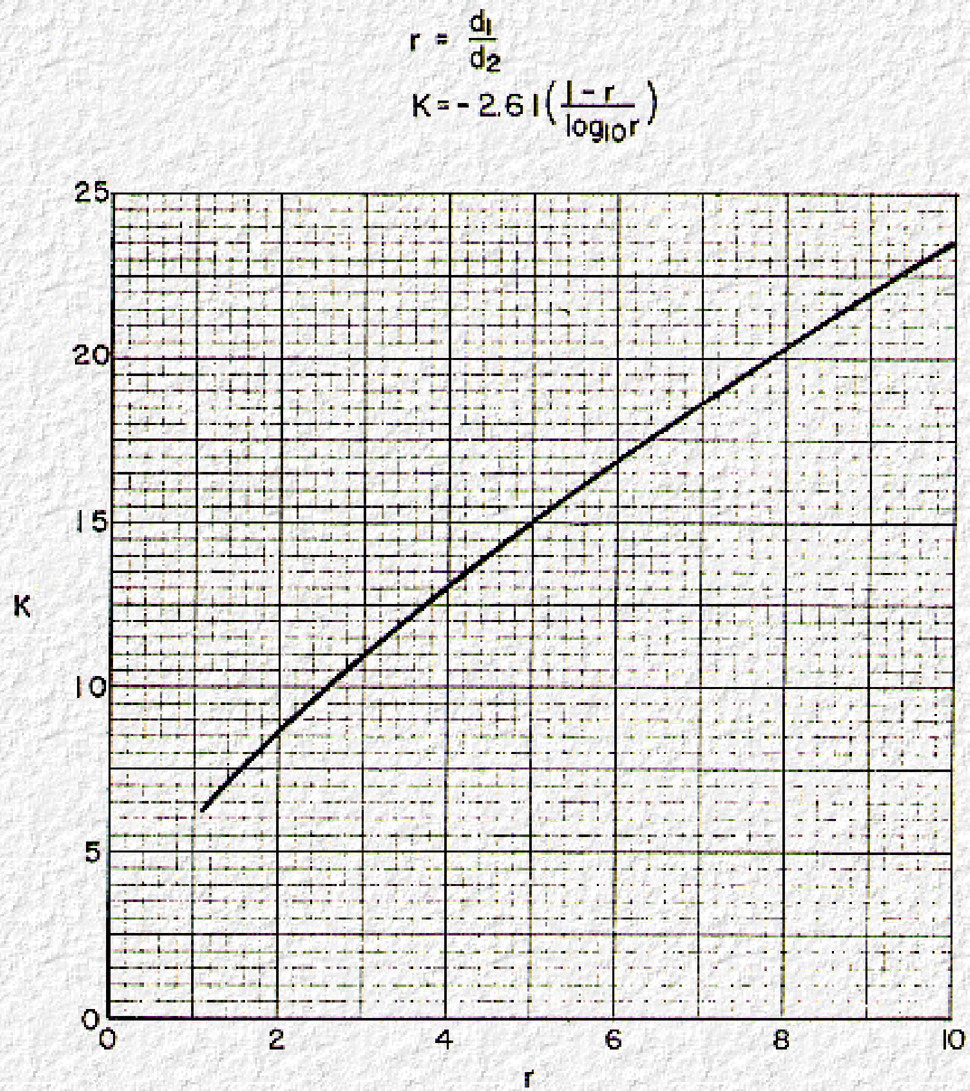


Figure 2a. K vs. r for r < or = 10

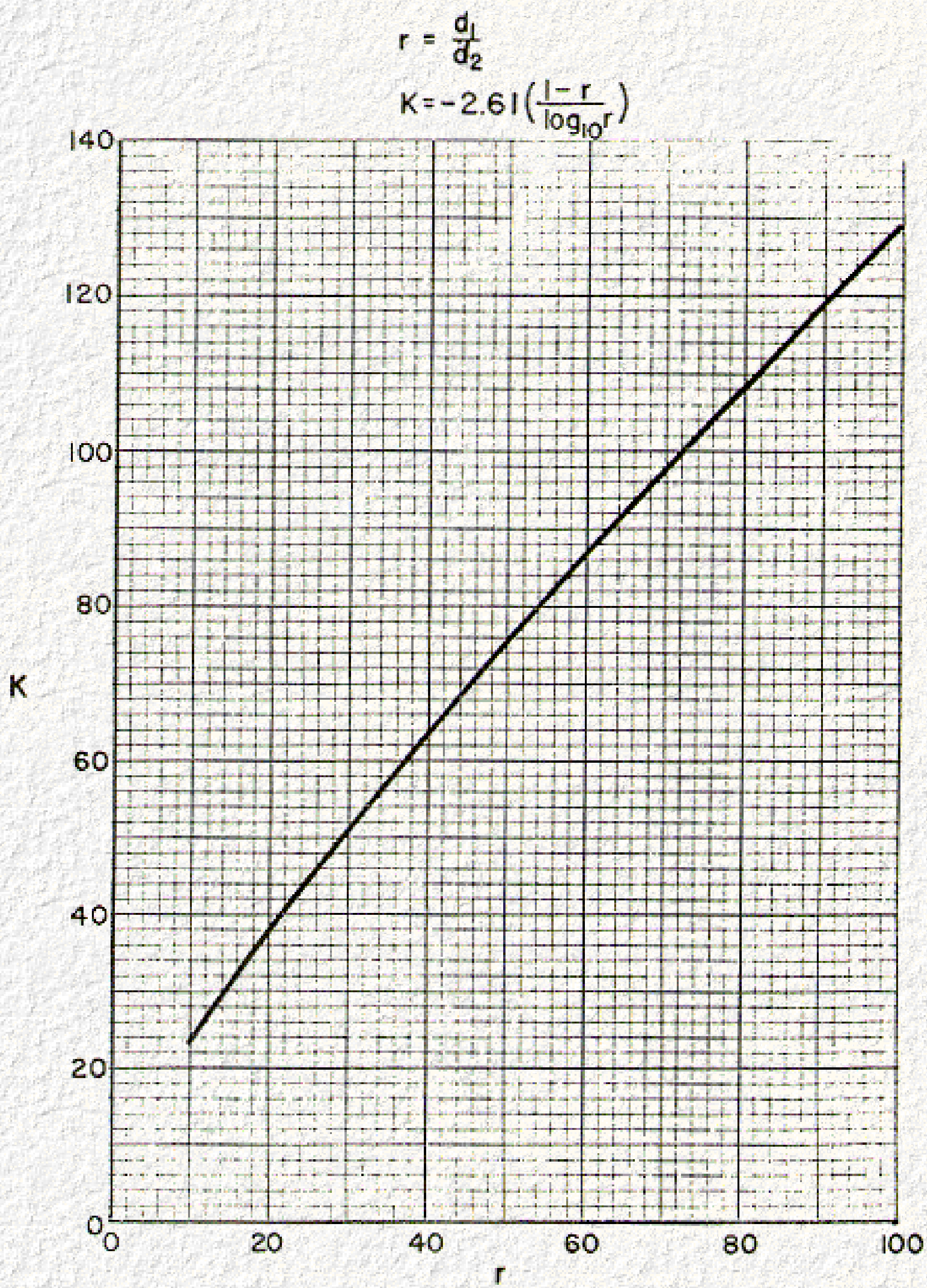


Figure 2b. K vs. r for 10 < or = r < or = 100

6.3 Computer Solution

If a computer terminal is available, it can be used to compute the specific surface utilizing the program contained in Appendix C. Figure 3 shows the input for and the output from a computer solution of specific surface.

The input data consists of:

a. One line of 30 characters (alphabetic or numerical) to identify the sample.

b. Up to fifteen lines of grain-size analysis and shape factor data. Each line consists of the following three pieces of information separated by commas:

1) The opening size, in centimeters, of the finer sieve in a sieve Interval or d_2 (Column on Form SM 389).

2) The decimal fraction of the total specimen in that sieve interval (Column 5 on Form SM 389).

3) The shape factor (Column 6 on Form SM 389).

The first of these lines should contain the size of the largest particle in the specimen or the opening size of the finest sieve in the sieve series through which the entire specimen passed. This should be followed by 0.0, 0.0, for percent in sieve interval and shape factor.

The output consists of:

a. The first input line (sample identification).

b. Three columns:

1) A column, headed D_1, showing the opening size, in centimeters, of the coarser sieve in each sieve interval.

2) A column, headed D_2, showing the opening size, in centimeters, of the finer sieve in each sieve interval.

3) A column, headed SSI, giving the corrected specific surface for each sieve interval.

c. The total specific surface (TOTAL SAMP SPEC SURF) of solids for one cubic centimeter of solids of the material.

```
SAMPLE PROBLEM
1.91,Ø.Ø,Ø.Ø
.951,.Ø64,1.15,
.476,.114,1.2,
.238,.Ø962,1.25,
.119,.1422,1.3,
.Ø595,.152,1.45,
.Ø297,.2524,1.45,
.Ø149,.121,1.5,
.ØØ74,.Ø16,1.5,
.ØØ18,.Ø422,1.6,
```

SAMPLE PROBLEM

	D1	D2	SSI
1	1.91ØØ	Ø.951Ø	Ø.33
2	Ø.951Ø	Ø.476Ø	1.25
3	Ø.476Ø	Ø.238Ø	2.19
4	Ø.238Ø	Ø.119Ø	6.73
5	Ø.119Ø	Ø.Ø595	16.Ø6
6	Ø.Ø595	Ø.Ø297	53.38
7	Ø.Ø297	Ø.Ø149	52.89
8	Ø.Ø149	Ø.ØØ74	14.Ø2
9	Ø.ØØ74	Ø.ØØ18	12Ø.67

TOTAL SAMP SPEC SURF = 267.52

Figure 3. Input for (above) and Output From (below) the Computer Solution

---

Appendix A : APPLICATION OF RESULTS

Determination of Saturated Permeability - The saturated permeability of the cohesionless granular material con be determined from the formula:

$$\log_{10}k_s = 1.365 = 5.15\ n - 2\log_{10}S$$

where, ks = saturated permeability in cm/sec, S = Specific surface of solids in cm^2/cm^3

n = porosity

The permeability determined by means of this formula will be quite accurate for the specimen at various values of porosity. The result, however, may differ from the actual field permeability, even though the value for the specimen may be correct. Disparities may result for the following reasons:

. Because of the natural variability of the soil, the sample may not be representative of the soils actually present. In order to minimize this error, a sufficient number of samples should be obtained to adequately investigate the foundation soils.

b. There may be difficulties in determining field in-place porosity. The in-place porosity may be estimated on the basis of the number of blows per foot of sampler spoon penetration. The conversion from blows per foot to porosity may be made through the use of Page 4-7 in New York State Department of Public Works, Bureaus of Soil Mechanics Design Reference Book, 1964 Edition, and Figure 3-8 in Navdocks DM-7, Design Manual, 1961. When sampling cohesionless soils below the water table, the drill hole should be kept filled with water to above the surrounding groundwater level during the entire operation, including withdrawal of the rods. Otherwise, a quick condition is caused at the bottom of the hole, loosening the soil, and greatly reducing the penetration resistance. As a result, the porosity would be over-estimated and an excessively high permeability calculated.

c. The relationship between specific surface and permeability will be more complex in skip-graded materials composed of coarse particles "floating" in a relatively fine-grained matrix. In this case, the addition of the coarse particles to the matrix reduces the specific surface of the combined soil. However, since the spaces between them are filled with the fine-grained matrix, the diameter of the average void does not change, but the total volume of voids is reduced. For a skip-graded soil, the permeability of the matrix can be established on the basis of the specific surface of the fine-grained material. This permeability should then be reduced by the ratio of the volume of the coarse inclusions to the volume of the matrix in order to arrive at an estimate of the permeability of the mixture.

d. Sedimentary soils are generally deposited in horizontal layers, the gradations of which can vary within very small vertical distances. Consequently, the horizontal permeability can be much greater than the vertical permeability in the field. Therefore, in cases where effects of such stratification are important, field permeability tests provide a more reliable value of the actual field permeability.

Other Uses - It is probable that the specific surface of a granular material, when adequately correlated with other properties, with a close theoretical relationship, can be put to uses other than the determination of saturated permeability, such as:

a. Determination of capillary rise and capillary suction potential of non-plastic soils.

b. Determination of transmission zone water content for unsaturated non-steady state flow involving non plastic soils, such as for recharge basin design.

c. Determination of the optimum amount of various asphalts for an asphalt-stabilized material.

Appendix E : SAMPLE PROBLEM

In this sample problem the specific surface has been computed for a granular material with the gradation and the shape factors shown on the attached grain-size distribution chart and recorded in the completed Form SM 389. The specific surface of the sample has been computed arithmetically on Form SM 389. The input and output of the computer solution are also attached.

The total specific surface of 267.78 cm^2/cm calculated arithmetically is in good agreement with the total sample specific surface of 267.52 cm^2/cm obtained by use of the computer.

SM 389a (9/73)

NEW YORK STATE - DEPARTMENT OF TRANSPORTATION

SPECIFIC SURFACE OF GRANULAR MATERIALS

P.I.N. ______

PROJECT Sample Problem

DRILL HOLE NO. ______ SAMPLE NO. ______ DEPTH ______

REGION ______ COUNTY ______

TESTED BY ______ REGION/MAIN OFFICE DATE ______

	SAMPLE INFORMATION				MOISTURE CONTENT		
A	Wt. + 3/8" Dry	Lbs.	0.31	J	Tare No.		13
B	Wt. - 3/8" Moist	Lbs.	4.71	K	Wt. Tare & Wet Soil	Gms	514.6
C	Wt. - 3/8" Dry (B ÷ (1+R))	Lbs.	4.54	L	Wt. Tare & Dry Soil	Gms	496.2
D	Wt. Total Specimen Dry (A+C)	Lbs.	4.85	M	Wt. Tare	Gms	35.5
E	Fraction Ret. on 3/8" (A÷D)		0.064	N	Wt. of Water (K-L)	Gms	18.4
F	Fraction Passing 3/8" (1-E)		0.936	P	Wt. Dry Soil (L-M)	Gms	460.7
G	Dry Wt. - 3/8" Before Wash	Gms	500.0	R	% Moisture ((N÷P) x 100)		4.0 %
H	Dry Wt. - 3/8" After Wash	Gms	288.5				
I	Wt. Mat'l Lost in Wash (G-H)	Gms	11.5				

1	2	3	4	5	6	7	8	9	10	11	12
										SPECIFIC SURFACE	
Sieve Des.	Opening Size (CM)	W.T. Retained	Fraction Ret. [Partial] NOTE 2	Fraction Ret. [Total] NOTE 3	Shape Factor	d_1 (CM) NOTE 4	$\frac{d_1}{d_2}$ NOTE 4	K [From Fig.2]	$(Ss)_{av}$ $\frac{(CM^2)}{(CM^3)}$ 9 ÷ 7	Spherical (CM^2/CM^3) 5 x 10	Corrected (CM^2/CM^3) 11 x 6
3"	7.61					7.61	2.0	8.7	1.14		
1-1/2"	3.81					3.81	2.0	8.7	2.28		
3/4"	1.91	0	0	0		1.91	2.0	8.7	4.55	0.29	0.33
3/8"	0.951	0.31 lbs.	1.0	0.0640	1.15	0.951	2.0	8.7	9.15	1.04	1.25
#4	0.476	36.6 gms	0.1220	0.1140	1.20	0.476	2.0	8.7	18.3	1.76	2.20
#8	0.238	30.9	0.1030	0.0962	1.25	0.238	2.0	8.7	36.6	5.20	6.76
#16	0.119	45.6	0.1520	0.1422	1.30	0.119	2.0	8.7	73.1	11.11	16.11
#30	0.0595	48.7	0.1623	0.1520	1.45	0.0595	2.0	8.7	146	36.91	53.51
#50	0.0297	80.9	0.2697	0.2524	1.45	0.0297	2.0	8.7	293	35.44	53.17
#100	0.0149	38.8	0.1293	0.1210	1.50	0.0149	2.0	8.7	584	9.34	14.01
#200	0.0074	5.1	0.0170	0.0160	1.50	0.0074	4.1	13.2	1784	75.28	120.44
		Pan plus mat'l. lost in	0.0450	0.0422	1.60						

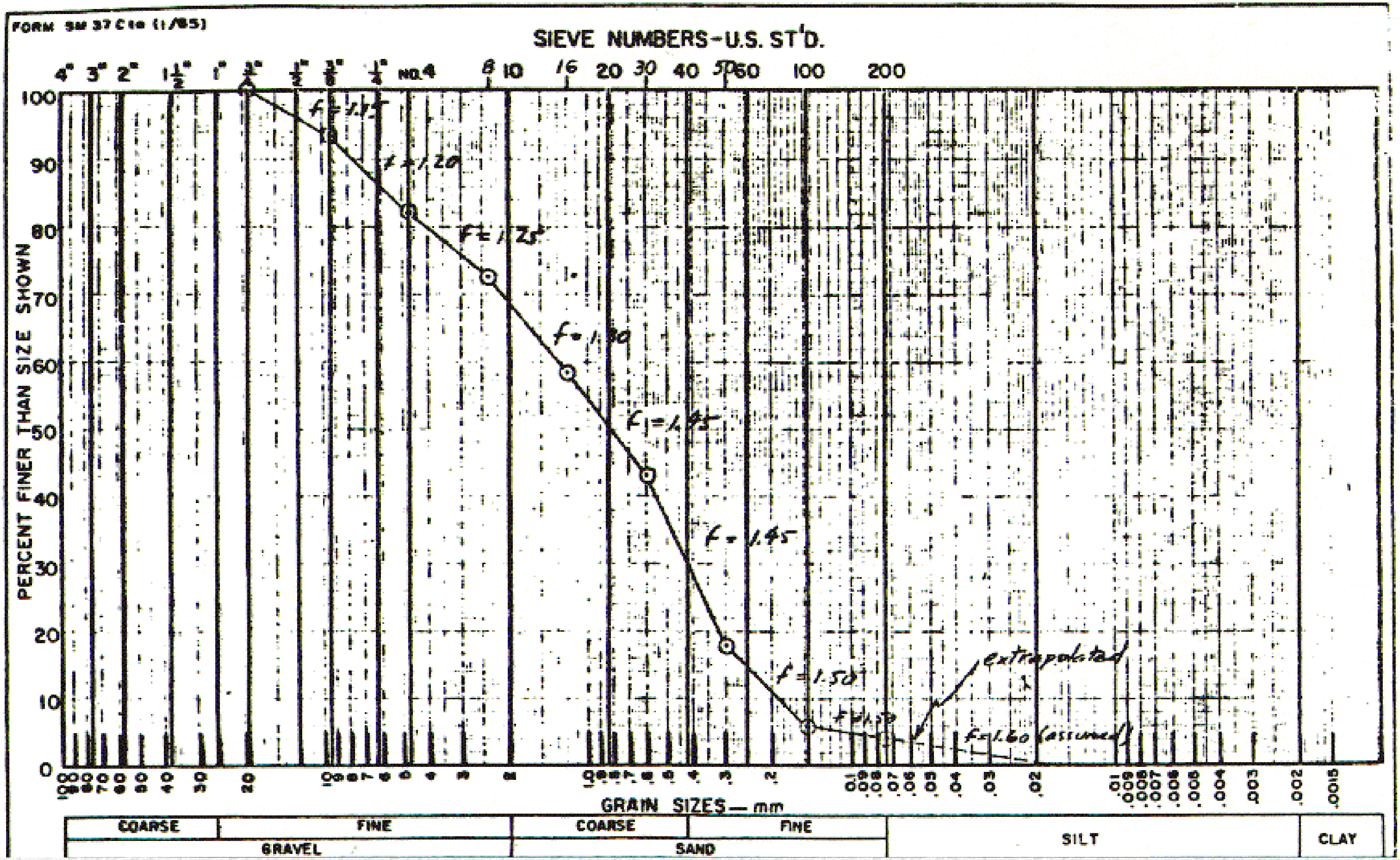
		wash: 13.5 gm								

CHECK TOTAL (=1.000) 1.0000

TOTAL SPECIFIC SURFACE 267.78

NOTES

1. See manual for method of obtaining grain-size distr. for mat. passing the #200 sieve.
2. For plus 3/8-in. specimen, 4 = 3 + A
   For minus 3/8-in. specimen, 4 = 3 + 0
3. For plus 3/8-in. specimen, 5 = 4 x E
   For minus 3/8-in. specimen, 5 = 4 x F
4. d_1 = opening size of upper sieve in a sieve interval.
   d_2 = opening size of lower sieve in a sieve interval.



PROJECT SAMPLE PROBLEM	STATE OF NEW YORK
SAMPLE NO. ___ DISTRICT NO. ___ COUNTY ___	DEPARTMENT OF PUBLIC WORKS
STATION ___ OFFSET ___ DEPTH ___	DIVISION OF CONSTRUCTION
DATE ___ DRAWN BY ___	BUREAU OF SOIL MECHANICS
	GRAIN SIZE DISTRIBUTION CURVE

```
B-5500 01/12
?LI:B0T0:6D6B092
#STATION 1/12 LOGGED-IN AT 1508 BY B0T0
?HM

B0STATION NOT ASSIGNED??
??RUN UDATCOM/HANDLER

 1:UDATCOM/HANDLER= 1 RUNNING 1509
B0T0 16
ENTER PROGRAM ID

S560010 B0TE10455701000E3651
ENTER DATA
TAPE

TAPE OK

SAMPLE PROBLEM
1.91,0.0,0.0,
.951,.064,1.15,
.476,.114,1.2,
.238,.0962,1.25,
.119,.1422,1.3,
.0595,.152,1.45,
.0297,.2524,1.45,
.0149,.121,1.5,
.0074,.016,1.5,
.0018,.0422,1.6,
EOT
OK
RUN

REQUEST SCHEDULED


?TS
 3:S560010/HANDLER=01 IN FOR 1:34, NEEDS 4992
 3:SOUNDS/HANDLER=02 IN FOR 19, NEEDS 5632
?MX
 1:UDATCOM/HANDLER= 1
```

NYSDOT SPECIFIC SURFACE ANALYSIS 3:15 PM 19 DEC 1972

SAMPLE PROBLEM

	D1	D2	SSI
1	1.9100	0.9510	0.33
2	0.9510	0.4760	1.25
3	0.4760	0.2380	2.19
4	0.2380	0.1190	6.73
5	0.1190	0.0595	16.06
6	0.0595	0.0297	53.38
7	0.0297	0.0149	52.89
8	0.0149	0.0074	14.02
9	0.0074	0.0018	120.67

TOTAL SAMP SPEC SURF = 267.52

D.3 Laboratory Permeability Methods

---

1. Constant-Head Test

In the constant-head test water flows through a test specimen under a measured hydraulic gradient (a constant amount of head is maintained) and through a known cross-sectional area (See Figure D-3-1). Coefficient of permeability is calculated from Darcy's law arranged in the form:

$$k = \frac{q}{iA} = \frac{Q}{iAt} \qquad \text{(D-3.1)}$$

In Equation D-3.1, k is the calculated coefficient of permeability, Q is the total seepage quantity flowing in time t, q = Q/t = the rate of quantity per unit of time, A is the cross-sectional area of test specimen (soil Sample) and i is the hydraulic gradient) the loss of hydraulic head per unit distance of flow, H/L.

> EXAMPLE: Assume that 20 cu ft (0.56 m^3) of water flows through a test specimen with a cross-sectional area A = 1.0 sq ft (0.093 m^2), under the hydraulic gradient of 0.8, in 24 hours.
>
> Then, k = q/iA = (20 cu ft/day)/(0.8 x 1.0 sq ft) = 25 ft/day (7.63m/day)

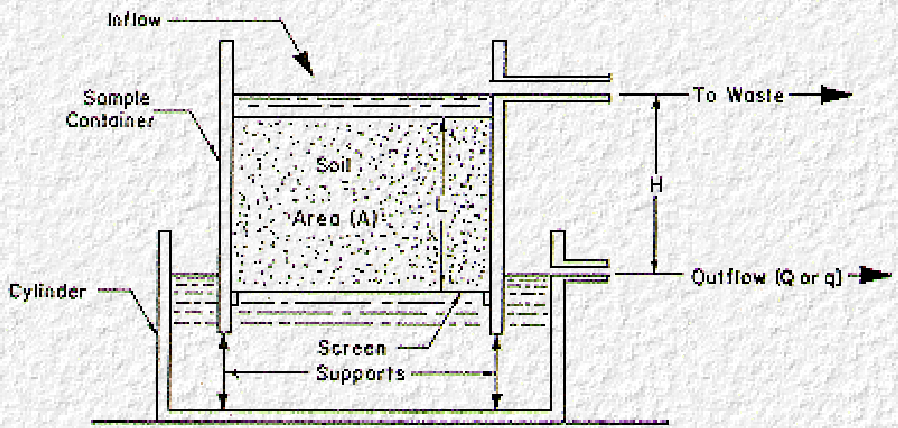


Figure D-3-1. Constant-Head Permeability Test

The constant-head test is used for determining the permeability of remolded samples of coarse-grained soils such as clean sands and gravels.

---

2. Falling-Head Test

In the falling-head test the amount of head inducing flow is allowed to decrease. By measuring the head at several time intervals in a small-diameter riser tube, while water is flowing through a soil specimen of greater cross-sectional area than the riser tube (see Figure D-3-2), coefficient of permeability is calculated from the formula:

$$k = \frac{2.3aL}{A\,dt}\log_{10}\frac{h_0}{h_1}$$

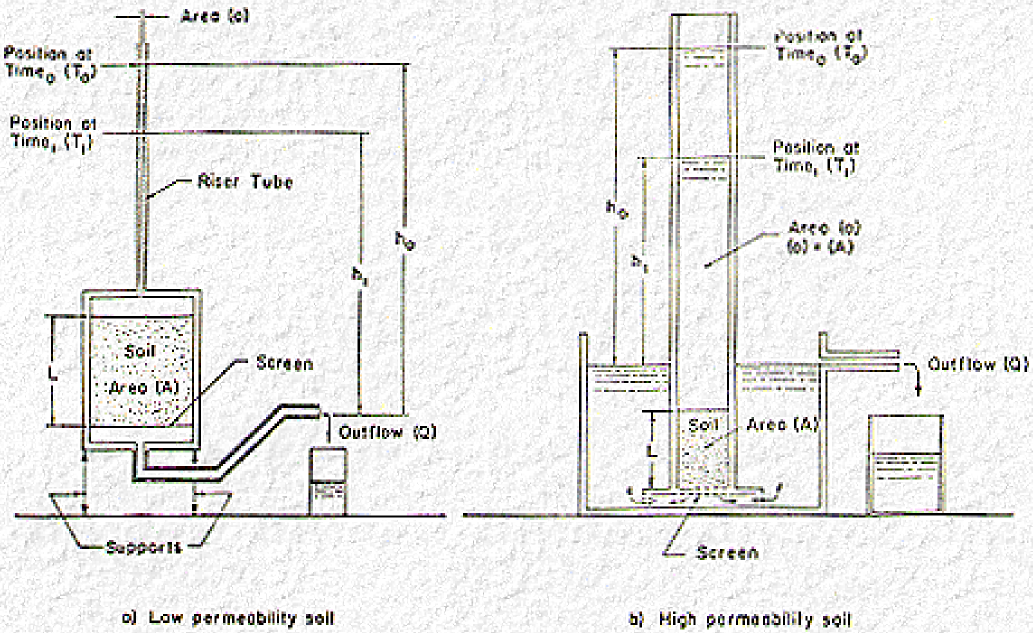


Figure D-3-2. Falling Head Head Permeability Test

In Equation D-3.2, a is the cross-sectional area of the riser tube, A is the cross-sectional area of the soil specimen, L is the length of the soil specimen, dt is the time interval during which the head drops from its initial value h_o to some lower value, h_1.

EXAMPLE: Referring to Figure D-3-2a, assume that h_0 = 3.0 ft (91 cm), h_1 = 2.5 ft (76 cm), dt = 6 hrs = 0.25 day, a = 0.01 sq ft (0.22 cm^2), A = 0.5 sq ft (465 cm^2), and L = 0.5 ft (15.24 cm). Using Equation D-3.2,

$$k = \frac{2.3aL}{A\,dt}\log_{10}\frac{h_0}{h_1} \qquad \text{(D-3.2)}$$

$$= \frac{2.3(0.01)(0.5)}{0.5(0.25)}\log_{10}\frac{3.0}{2.5}$$

= (0.092)(0.079) = 0.0073 ft/day

(25.6×10^{-7} cm/sec.)

Figure D-3-2a and the above calculation pertain to tests on low permeability fine grained materials. If the material being tested has moderately high permeability, a falling-head test may often be made with the arrangement shown in Figure D-3-2b, in which the cross-sectional area of the standpipe, a, is equal to the cross-sectional area of the sample, A, in Equation D-3.2. For very high permeability soils, the constant head test shown in Figure D-3-1 is used.

In all laboratory tests, adequate precautions should be taken to minimize experimental errors. Many soil mechanics text books give the details and precautions. Generally the test results are corrected to the viscosity of water at 20°C.

D.4. Field Permeability Method for Design of Basins Using Single Ring Test

Infiltration Test Specifications for Infiltration Basins with Low Water Tables (Contra Costa County Flood Control and Water Conservation District, California).

1. A 12-inch (0.304 m) diameter or larger steel pipe shall be driven into the ground a minimum distance of 12 inches (0.304 m).

2. The elevation of the ground within the pipe at the time of the test shall not vary more than one foot (0.3 m) from the final elevation of the infiltration basin.

3. A burlap sack or layer of gravel shall be placed on the soil surface within the pipe to avoid disturbance of the soil when water is poured in the pipe.

4. Water shall be put in the pipe to a depth of at least 6 inches (0.15 m) and the depth shall not exceed the depth of water in the final design of the infiltration basin. Water shall be added to the pipe when necessary to insure the ground within the pipe is always covered during the test.

5. Time measurements shall be taken for various drops in water surface within the pipe. Attached is a sample of the Infiltration Test Form (Figure D-4-1).

Single Ring Infiltration Test

Tract No. ____________________ Date ____________________

Location ____

Test No. ______________ Soil Type ______________ Test By ______________

Time	Elapsed Time Since Last Reading (Minutes)	Depth To Water (Feet)		Intake During Period (Minutes)	Infiltration Rate Per Hour		Remarks
		Before Filling	After Filling		(Feet)	(Inches)	

Figure D-4-1. Data Recording Sheet used by Contra Costa County, Calif. For Single Ring Infiltration Tests

6. The Infiltration Rate Curve shall be drawn using Accumulated Time from Beginning of Test in minutes as the abscissa and Infiltration Rate in inches per hour (mm/hr) as the ordinate. Refer to the typical plot on Figure D-4-2.

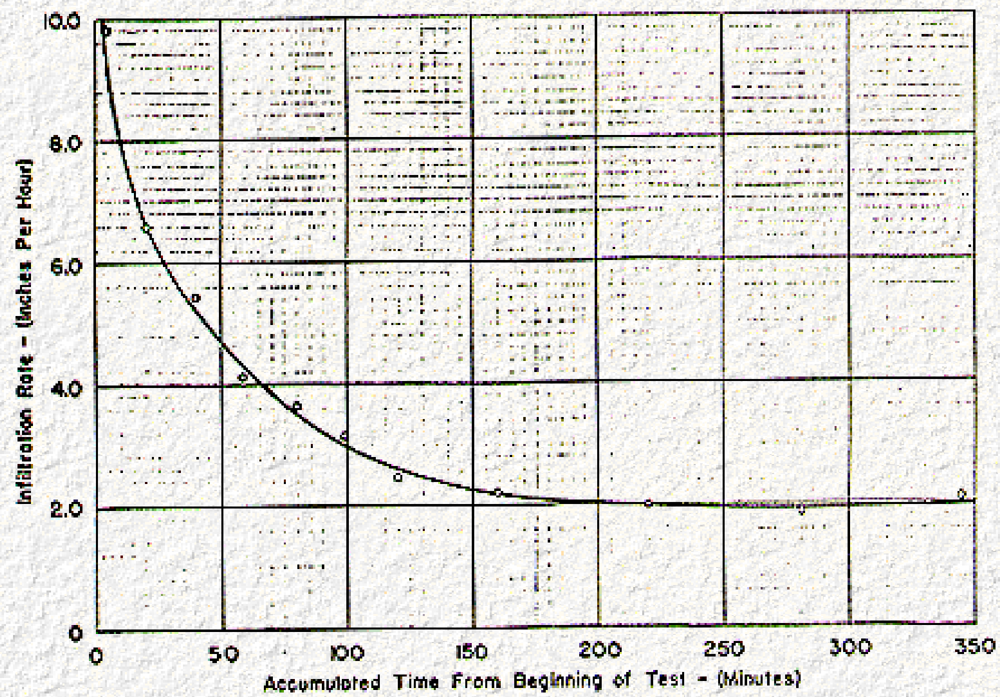


Figure D-4-2. Typical Plot of Test Data for Single Ring Infiltration Test (Contra Costa County, Calif.)

7. Each infiltration test shall be continuous and of sufficient duration such that the last three computed infiltration rates do not vary from each other by more than 5%.

8. A minimum of three infiltration tests shall be made for each infiltration basin and the locations of the tests shall be shown on a map.

9. The infiltration tests shall be performed under the direction of and certified by a civil engineer registered in the State of California.

10. If sufficient soil information is not available, additional soil information will be required to be furnished. This soil information shall be of the field test nature by a qualified soils technician. Depths from ground surface to beginning and end of each soil characteristic or texture shall be noted. The depth of the test hole shall not be less than six feet (1.83 m) and shall go to a depth sufficient to determine there is no layer restricting permeability.

D.5. Field Permeability Method for Design of Basins Using Double Ring Test

Double Ring Method to Estimate Infiltration from Basins with Low Water Tables

At a site where an infiltration basin is planned (see Figure D-5-1a) double-ring infiltrometer tests can be made at several locations with rings set to the planned bottom of the basin, as shown. If downward flow in the inner ring is essentially parallel, the measured infiltration rate I, can be used in estimating the drainage capability of the basin (as long as the water table is deep and the controlling flow is downward seepage, as shown in Figure D-5-1b. For this example, the capability of the basin to discharge seepage is Q = IA, with Q being the capacity per day, I the vertical infiltration rate determined with the double ring infiltrometer, and A the area of the basin.

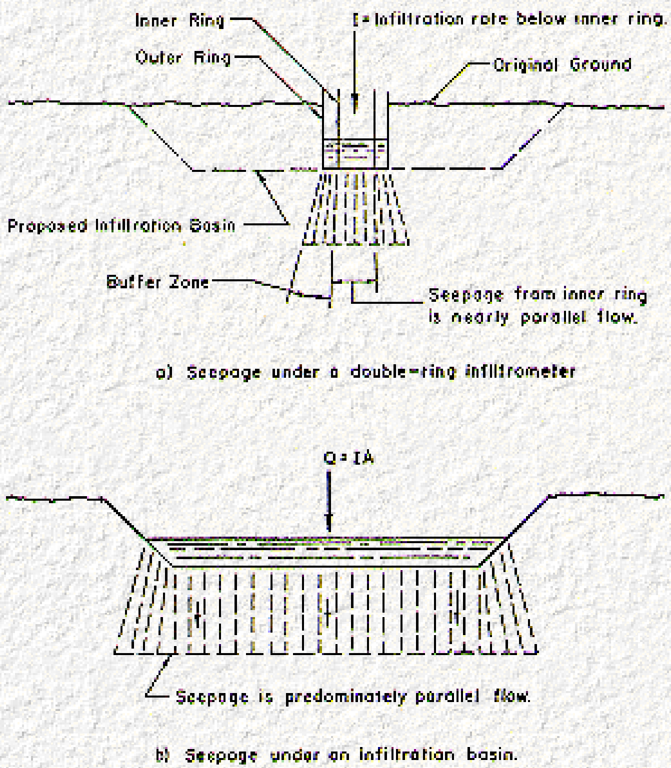


Figure D-5-1. Double-Ring Infiltrometer to Estimate Seepage from Infiltration Basin

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D.6. Auger Hole Tests

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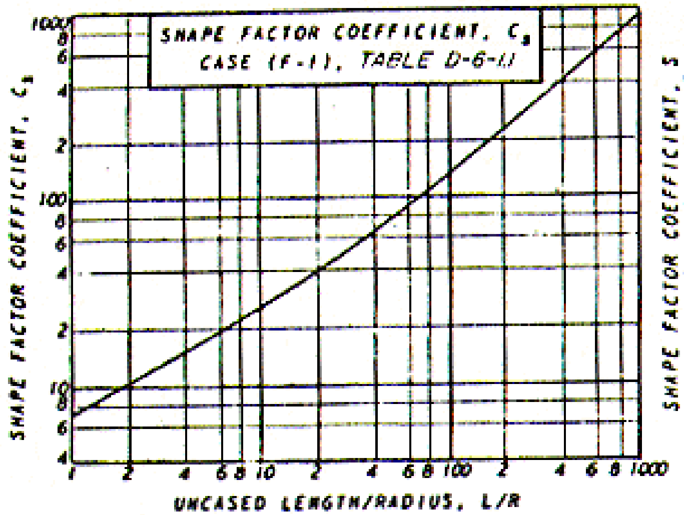
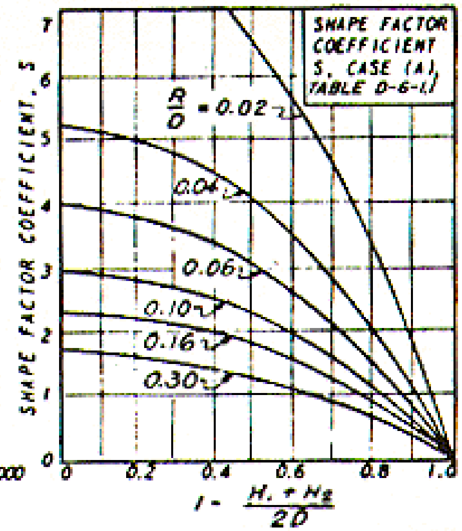
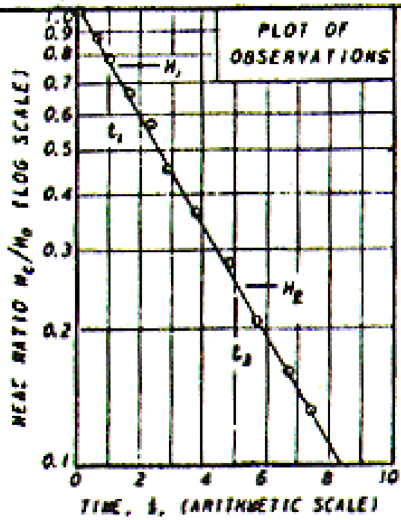
1. Variable Head Permeability Test

At selected points located within the limits of a proposed infiltration system, holes 9-inches (229 mm) in diameter or larger are bored to at least 2.5 feet (0.76 m) below the low-water elevation expected at the site, or at least 2.5 feet (0.76 m) below the existing water table, whichever is lower. The bottom 2.5 feet (0.76 m) of the hole must be kept open during a test, and if this cannot be accomplished with open auger holes, a 2-inch (50± mm) diameter wellpoint is put down or a cased hole is used. These tests, which may be called "variable head permeability tests" (U.S. Dept. of the Navy, Naval Facilities Engineering Command, 1974), offer a way of testing for the in situ permeabilities of soil formations. In making a test, the water level is either raised or lowered from the equilibrium level and allowed to recover while readings are made of water elevations versus elapsed time. Figure D-6-1.1 gives methods for analyzing the information. Table D-6-1.1 shows how to compute permeability using shape factors for various test configurations. If soils are anisotropic, a method for transforming the dimensions of the intake point of the piezometer or observation well is described in Figure D-6-1.1. The Navy suggests that one "Assume various ratios of horizontal to vertical permeability until mean permeability determined from several piezometers is made equal". If tests are made in open-end or uncased boreholes, procedures and methods may be used as outlined in Table D-6-1.2.



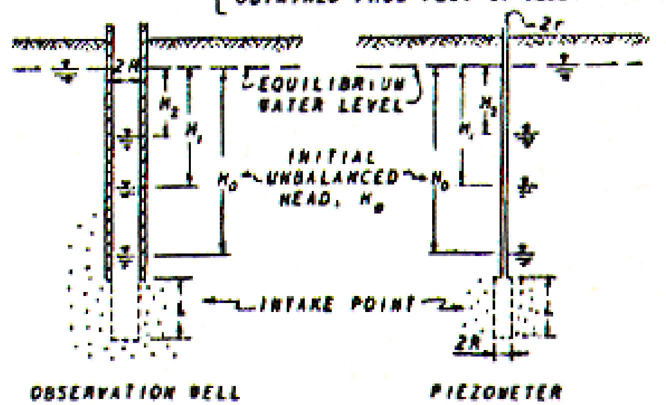




IN GENERAL:

$$K = \frac{A}{F(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$$

F = SHAPE FACTOR OF INTAKE POINT
A = STANDPIPE AREA
K = MEAN PERMEABILITY
Ln H_1/H_2 AND $(t_2 - t_1)$ ARE OBTAINED FROM PLOT OF OBSERVATIONS.



OBSERVATION WELL IN ISOTROPIC SOIL:

OBTAIN SHAPE FACTOR FROM TABLE D-6-1.1. FOR CASE (C):

$$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$$

$$K = \frac{R^2}{2L} \ln\left(\frac{L}{R}\right)\left[\frac{\ln H_1/H_2}{(t_2 - t_1)}\right]$$

PIEZOMETER IN ISOTROPIC SOIL:

RADIUS OF INTAKE POINT (R) DIFFERS FROM RADIUS OF STANDPIPE (r).

$$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$$

$$A = \pi r^2$$

$$K = \frac{A}{F(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$$

$$K = \frac{r^2}{2L} \ln\left(\frac{L}{R}\right)\left[\frac{\ln H_1/H_2}{t_2 - t_1}\right]$$

TEST IN ANISOTROPIC SOIL:

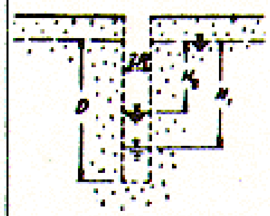
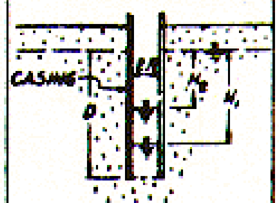
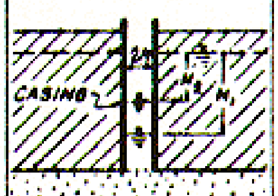
ESTIMATE RATIO OF HORIZONTAL TO VERTICAL PERMEABILITY AND DIVIDE HORIZONTAL DIMENSIONS OF THE INTAKE POINT BY: $m = \sqrt{K_H/K_V}$ TO COMPUTE MEAN PERMEABILITY $K = \sqrt{K_H \times K_V}$.

FOR CASE (C), TABLE D-6-1.1:

$$F = \frac{2\pi L}{\ln\left(\frac{mL}{R}\right)}$$

$$K = \frac{r^2}{2L} \ln\left(\frac{mL}{R}\right) \frac{\ln H_1/H_2}{(t_2 - t_1)}$$

Figure D-6-1.1 Analysis of Permeability by Variable Head Tests.*(Navy, 1974)*

	Condition	Diagram	Shape Factor, F	Permeability, K by variable head test	Applicability
OBSERVATION WELL OR PIEZOMETER IN SATURATED ISOTOPIC STRATUM OF INFINITE DEPTH	(A) Uncased hole		$F = 16\pi DSR$	(for observation well of constant cross section) $K = \frac{R}{16DS} \times \frac{(H_2 - H_1)}{(t_2 - t_1)}$ FOR $\frac{D}{R} < 50$	Simplest method for permeability determination. Not applicable in stratified soils. For values of S see Fig. D-6-1.1.
	(B) Cased hole, soil flush with bottom.		$F = \frac{11R}{2}$	$K = \frac{2\pi R}{11(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$ FOR $6" \leq D \leq 60"$	Used for permeability determination at shallow depths below the water table. May yield unreliable results in falling head test with silting of bottom of hole.
	(C) Cased hole, uncased or perforated extension of length "L".		$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$	$K = \frac{R^2}{2L(t_2 - t_1)} \ln\left(\frac{L}{R}\right) \ln\left(\frac{H_1}{H_2}\right)$ FOR $\frac{L}{R} > 8$	Used for permeability determinations at greater depths below water table.
	(D) Cased hole, column of soil inside casing to height "L".		$F = \frac{11\pi R^2}{2\pi R + 11L}$	$K = \frac{2\pi R + 11L}{11(t_2 - t_1)} \ln \frac{H_1}{H_2}$	Principal use is for permeability in vertical direction in anisotropic soils.
...OMETER IN AQUIFER ...PER LAYER	(E) Cased hole, opening flush with upper boundary of aquifer of infinite depth.		$F = 4R$	$K = \frac{\pi R}{4(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	Used for permeability determination when surface impervious layer is relatively thin. May yield unreliable results in falling head test with silting of bottom of hole.
	(F) Cased hole, uncased or perforated		(1)	$K = \frac{\pi R}{\ldots} \ln\left(\frac{H_1}{\ldots}\right)$	Used for permeability determinations at

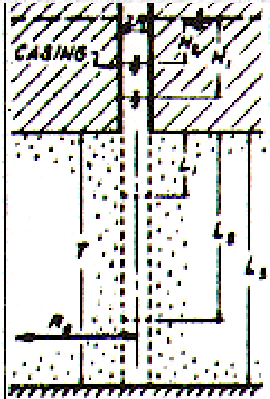
OBSERVATION WELL OR PIEZC WITH IMPERVIOUS UP[…]	extension into aquifer of finite thickness: (1) $\frac{L_3}{T} \leq 0.20$ (2) $0.2 < \frac{L_3}{T} < 0.85$ (3) $\frac{L_3}{T} = 1.00$ Note: R_o equals effective radius to source at constant head.		$F = C_S R$	$K = \ldots \frac{\ldots}{(t_2 - t_1)} \ldots \ln \frac{H_1}{H_2}$	depths greater than about 5 ft. For values of C_S see Fig. D-6-1.1.
			(2) $F = \frac{2\pi L_2}{\ln(L_2/R)}$	$K = \frac{R^2 \ln\left(\frac{L_2}{R}\right)}{2L_2(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$ for $\frac{L_2}{R} > 8$	Used for permeability determinations at greater depths and for fine grained soils using porous intake point of piezometer.
			(3) $F = \frac{2\pi L_3}{\ln\left(\frac{R_0}{R}\right)}$	$K = \frac{R^2 \ln\left(\frac{R_0}{R}\right)}{2L_3(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	Assume value of $\frac{R_o}{R}$ = 200 for estimates unless observations wells are made to determine actual value of R_o.

Table D-6-1.1 Shape Factors for Computation of Permeability from Variable Head Tests *(Navy, 1974)*

Property determined and name of test	Test application	Test equipment and procedure
Permeability: Pumping test	Applied for evaluation of problem of large scale drawdown or dewatering in materials with substantial variation in permeability.	Rate at which water is pumped from a central test well is measured while the drawdown or radial lines extending from the well is observed in a series of piezometers or observation holes. Provide 3 to 5 observation wells spaced at increasing intervals along two radial lines separated by 90 degree central angle.
Variable head test	Used to obtain individual permeability determinations in piezometers. Performed in porous tube or wellpoint piezometers which have been placed in boreholes with their intake point isolated by impervious seal.	Standpipe water level is raised or lowered from its equilibrium position and readings are taken of water levels at periodic intervals as it returns to equilibrium. Observations of differential head and time elapsed are analyzed as shown in Figure D-6-1.1.
Borehole test	Used to determine permeability of individual strata penetrated by borings.	May be performed in a cased, open-end borehole or an uncassed borehole with double packers. Rate at which water flows out of the borehole under a constant head is measured. Use equipment and procedures of USBR Method E-18.
Auger hole test	Performed in shallow uncased auger hole. May be used in unsaturated material where the water	Rate at which water flows out of the uncased hole is measured maintaining a constant water level in the hole. Use equipment and

	table during testing is at a great depth below the base of hole.	procedures of USBR Method E-19.

Table D-6-1.2. Measurements of Soil Permeability in Situ (Navy, 1974)

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2. Auger Hole Percolation Test of U.S. Dept of Health, Education, and Welfare

This method is described in Public Health Service Publication No. 526, "Manual of Septic-Tank Practice". It has been modified as presented below for use in determining infiltration rates for design of basins or other infiltration systems.

Percolation Tests

These percolation tests can be used to determine the acceptability of the site and establish the design size of the subsurface disposal system. The length of time required for percolation tests will vary in different types of soil. The safest method is to make tests in holes that have been kept filled with water for at least 4 hours, preferably overnight. This is particularly desirable if the tests are to be made by an inexperienced person. In some soils, such as those that swell upon wetting, it is necessary even if the individual has had considerable experience. Percolation rates should be figured on the basis of the test data obtained after the soil has had opportunity to become wetted or saturated and has had opportunity to swell for at least 24 hours. Enough tests should be made in separate holes to assure that the results are valid.

Procedure

1. Number and location of tests. - Six or more tests shall be made in separate test holes spaced uniformly over the proposed site.

2. Type of test hole. - Dig or bore a hole with horizontal dimensions of from 4 to 12 inches (0.1 to 0.3 m) and vertical sides to the depth of the proposed infiltration system. To minimize time, labor, and volume of water requirements per test, holes can be bored with a small hand auger.

3. Preparation of test hole. - Carefully scratch the bottom and sides of the hole with a knife blade or sharp-pointed instrument to disrupt any smeared soil surfaces and to provide a natural soil interface into which water may percolate. Remove all loose material from the hole. Add 2 inches (0.05 m) of coarse sand or fine gravel to protect the bottom from scouring and sediment (Fig. D-6-2.1.).

4. Saturation and swelling of the soil. -i It is important to distinguish between saturation and swelling. Saturation means that the void spaces between soil particles are full of water. This can be accomplished in a short period of time. Swelling is caused by intrusion of water into the individual soil particle. This is a slow process, especially in clay-type soil, and is the reason for requiring a prolonged soaking period.

In conducting the test, carefully fill the hole with clear water to a minimum depth of 12 inches (0.3 m) over the gravel. In most soils, it is necessary to refill the hole by supplying a surplus reservoir of water, possibly by means of an automatic siphon, to keep water in the hole for at least 4 hours and preferably overnight. Determine the percolation rate 24 hours after water is first added to the hole. This procedure is to insure that the soil is given ample opportunity to swell and approach the condition it will assume during the wettest season of the year. Thus, for a particular soil, the test will give comparable results whether made in a dry or in a wet season. In sandy soils containing little or no clay, the swelling procedure is not essential, and the test may be made as described under item 5C, below, after the water from one filling of the hole has completely seeped away. Refer to Figure D-6-2.1.

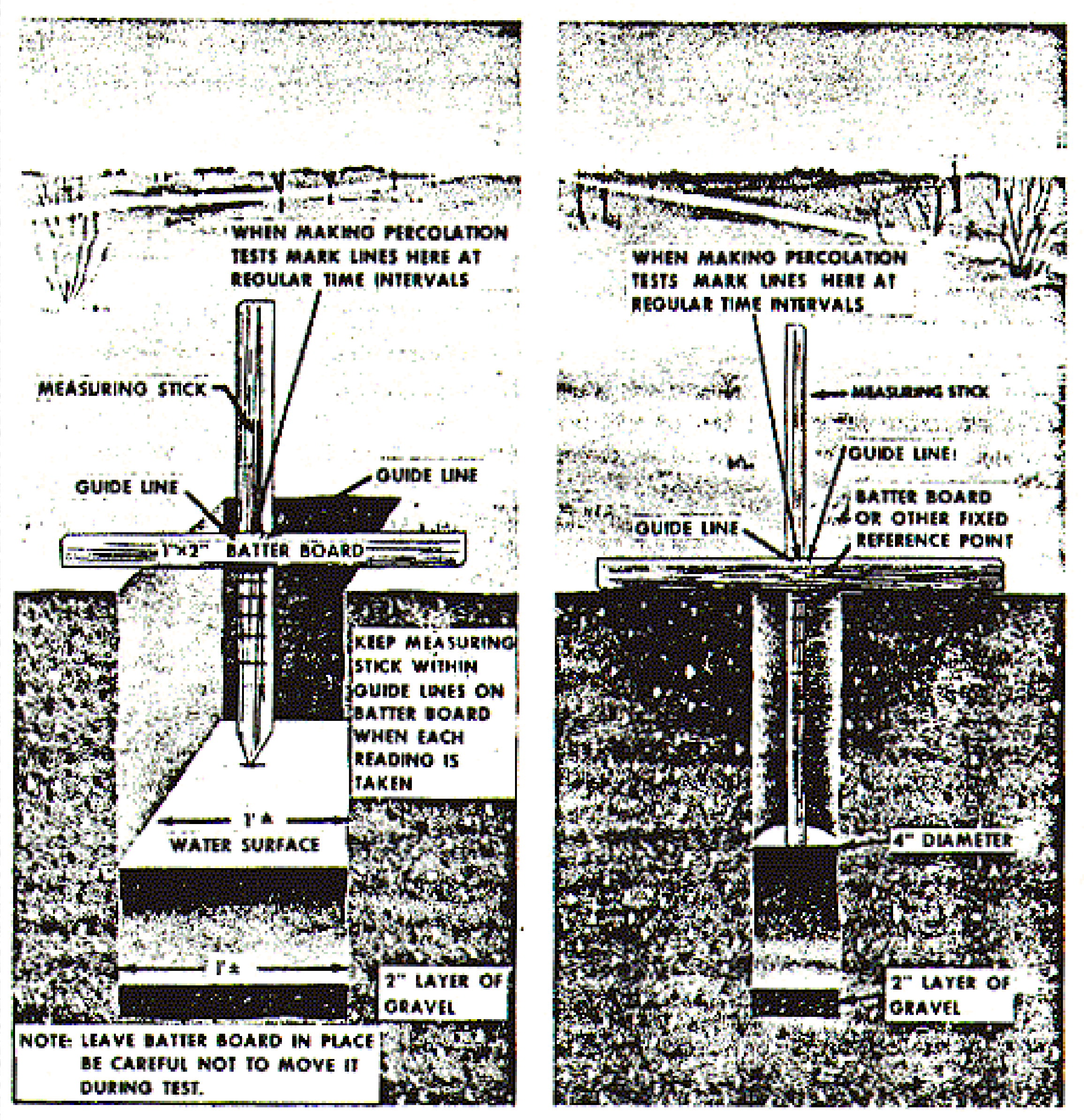

Figure D-6-2.1 Methods of Making Percolation Tests. ***(From U.S. Dept of HEW)***

5. Percolation-rate measurement. - With the exception of sandy soils, percolation-rate measurements shall be made on the day following the procedure described under item 4, above.

A. If water remains in the test hole after the overnight swelling period, adjust the depth to approximately 6 inches (0.15 m) over the gravel. From a fixed reference point, measure the drop in water level over a 30 minute period. This drop is used to calculate the percolation rate.

B. If no water remains in the hole after the overnight swelling period, add clear water to bring the depth of water in the hole to approximately 6 inches (0.15 m) over the gravel. From a fixed reference point, measure the drop in water-level at approximately 30 minute intervals for 4 hours, refilling 6 inches (0.15 m) over the gravel as necessary. The drop that occurs during the final 30 minute period is used to calculate the-percolation rate. The drops during prior periods provide information for possible modification of the procedure to suit local circumstances. -

C. In sandy soils (or other soils in which the first 6 inches (0.15 m) of water seeps away in less than 30 minutes, after the overnight swelling period), the time interval between measurements shall be taken as 10 minutes and the test run for one hour. The drop that occurs during the final 10 minutes is used to calculate the percolation rate.

D.7. Well Pumping Test for Design of Wells and Pits

A well pumping test is made by pumping water into or out of a well while the level of the water table within the influence of the test is being measured in nearby sounding wells or piezometers. During initial stages of a well pumping test, the water levels around a pumped well are lowering (toward an equilibrium condition) and useful solutions are available for estimating permeability during this time (Grover, 1966). The most common practice is to run a pumping test until equilibrium or a near equilibrium condition is reached and to calculate permeability from a steady seepage formula. The amount of time needed for obtaining useful permeability values from well pumping tests will vary with the permeability and thickness of the formations supplying the water, the areal extent of the aquifers, and the amount of water being pumped. In some cases, drawdown in sounding wells or piezometers installed near a pumping well will show little or no further changes after as little as an hour of steady pumping. In other cases, the drawdown will continue to change for several days. Frequently, continuous pumping with measurement of quantity pumped and drawdowns in a system of observation wells (sounding wells or piezometers) for a few hours up to one nominal 8-hr working day will suffice. If the size of a project warrants, or the equilibrium is slow in developing, continuous around-the-clock pumping and observing of drawdowns for two days or more may be warranted.

Glover, R. E. (1966), "Ground-Water Movement", Engineering Monograph No. 31, U.S. Dept. of the Interior, Bureau of Reclamation, Denver, Colorado, Second Printing, April 1966, pp. 4-16.

If a pumping well fully penetrates the primary water-supplying formation, the simple well formula can be used for calculating permeability.

For the case of radial flow to such a well, the Dupuit assumption provides the basis for deriving the formula for permeability. Dupuit assumed that the hydraulic gradient at any point is uniform from the top to the bottom of the water-supplying formation and is equal to the slope of the water surface above that point. Near a well, large errors are introduced but at moderate to large distances, they are relatively small. On the basis of this assumption the Darcy's law, permeability can be calculated with the following formula for the case of non-artesian or unconfined flow:

$$k = \frac{2.3q \log_{10}(r_2 / r_1)}{(h_2^{\,2} - h_1^{\,2})} \qquad \text{(D-7.1)}$$

In this equation, q is the steady state rate of inflow, h_2 is the thickness of the water-bearing formation at radial distance r_2 (observation well No. 2); h_1 is the thickness of the formation at distance r_1 (observation well No. 1), and k is the calculated coefficient of permeability of the water-bearing formation with a total thickness H. in Fig. D-7-1. Usually a number of calculations are made, using at least two pumping rates over a practical range.

Figure D-7-1 shows a plan view of a typical well pumping setup. The central pumping well should be large enough to insert a deep-well pump or other suitable pump capable of removing an amount of water (gpm) that will produce significant amounts of drawdown in the observation wells. A minimum of two observation wells in one radial line from the pumped well is needed for a permeability calculation. Generally three are installed in one radial line, at progressively larger distances (such as 5 ft. 10 ft. and 20 ft. or 10 ft. 20 ft. and 50 ft)(1.53, 3.05 and 6.1 m or 3.05, 6.1 and 15.25 m), and usually wells are installed in two or more radial lines surrounding a pumped well.

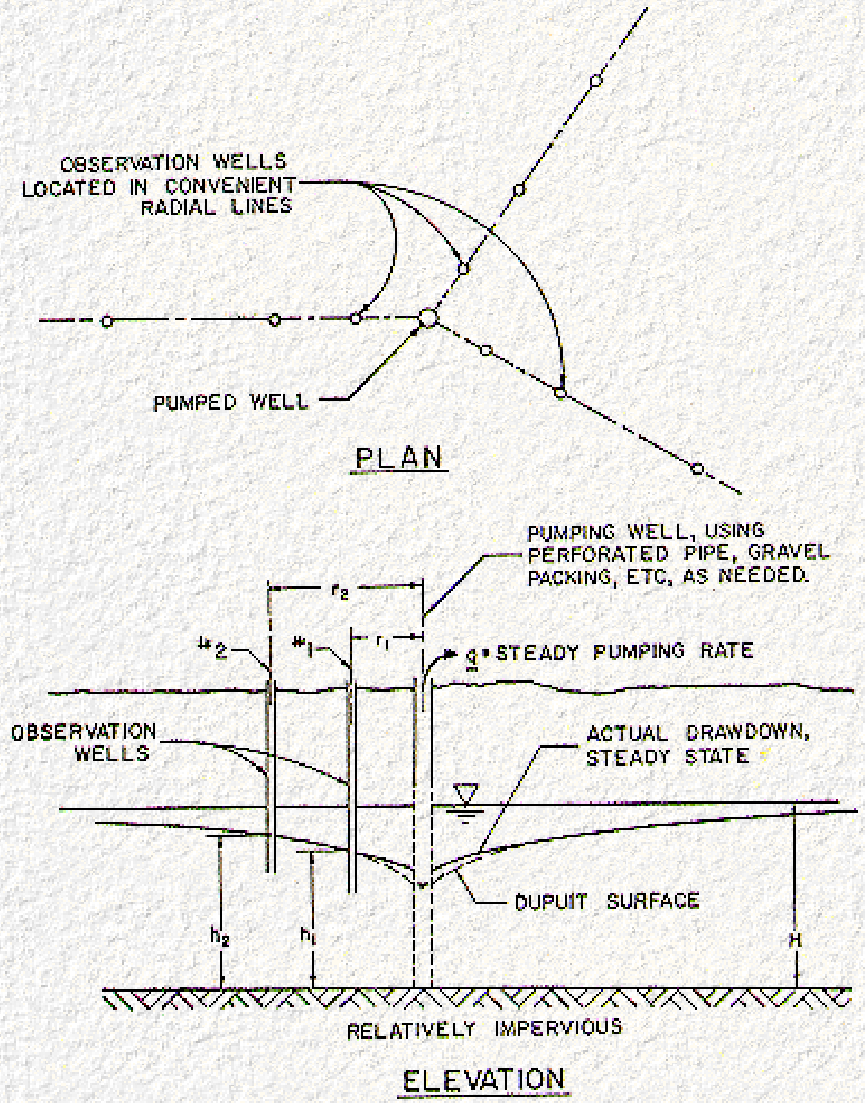

Figure D-7-1. A Typical Arrangement for a Well Pumping Test

D.8 Methods for Estimating Infiltration Rates

Using Darcy's Law and Flow Nets

As noted in Chapter 4-A-3 of the text, Darcy's law and flow nets are two useful methods for analyzing potential infiltration rates when the permeabilities of the formations are reasonably well known. Some theoretical solutions to seepage problems may be too rigorous for the assumptions that are made in the derivations and may fit actual cases only approximately. Most engineers experienced in seepage calculation feel that it is far better to make use of an approximate method than to rely on a rigorous theoretical formula that is not easily understood and which represent only a crude approximation of true conditions due to questionable assumptions. Some simplified procedures are shown in Figure D-8-1.

---

a. Vertical Flow Case

In Figure D-8-1a an infiltration basin is constructed in soil formations having a vertical unsaturated permeability of 1 ft/day (3.5 x 10^{-4} cm/sec), an impervious stratum at 20 ft depth (6.1 m), and a natural water table at 10 ft (3.05 m). What is the capability of the site for vertical discharge of water? From Darcy's law, $\underline{q} = \underline{kiA}$, the flow can be estimated from the vertical permeability of 1 ft/day (3.5 x 10^{-4} cm/sec), and a downward hydraulic gradient of 1.0 (conservative assumption), hence q = 1.0 ft/day (1.0) = 1.0 ft/day/sq ft (3.28 m/day/m^2).

For a 300-ft (91.5 m) wide basin, the possible $\underline{Q}$ is 300 cu ft/day/linear foot (27.54 m^3/day/m).

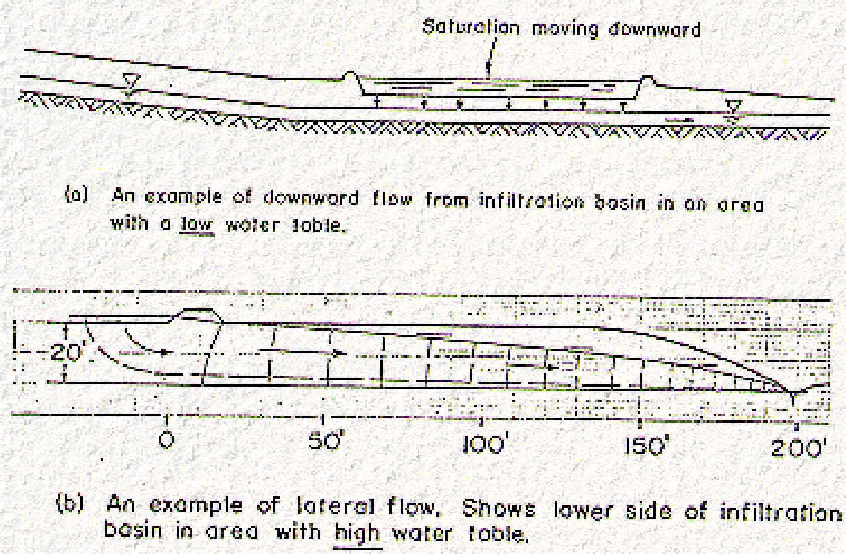

Figure D-8-1. Estimating Infiltration Rates with Darcy's Law and Flow Nets (Approximate Methods)(a) Downward Percolation (b) Lateral (Horizontal) Seepage

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b. Lateral Flow Case

Many sites that are capable of relatively large disposal rates when the underlying water table is low, become relatively valueless if the water table is high and flow is lateral. Figure D-8-1b illustrates the flow conditions at the edge of a basin where the groundwater mound has risen to the bottom of the basin. The flow net gives the shape factor, n_f/n_d, which is used in the following expression to estimate lateral seepage:

$$q = kh(n_f/n_d) \qquad \text{(D-8.1)}$$

Equation D-8.1 is the conventional formula for estimating seepage quantities with flow nets. The seepage quantity per linear foot is A, under net head h, while n_f is the number of flow channels in the flow net, and n_d is the number of equipotential drops. In this example, assuming a horizontal k = 5 ft/day (1.53 m/day), and a head of 20 ft (6.1 m), with the shape factor of n_f/n_d = 1.1/20, the seepage quantity is:

$q = kh(n_f/n_d)$ = (5 ft/day)(20 ft)(1.1/20) = 5.5 cu ft/day/foot (0.50 m^3/day/m)

If the flow is limited to only the low side of the infiltration basin (as assumed here) the potential for lateral flow of this basin is only about 2% of the vertical flow capability of the basin in Figure D-8-1a; even though the assumed lateral k is 5 times the vertical k in that example. When estimates of infiltration are made by the methods suggested here, designers should recognize they are approximate only and-should try to use conservative assumptions so as not to overestimate the capabilities of infiltration systems.

Go to Appendix E

Appendix E : FHWA-TS-80-218

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E.1 Proposed Standard Method of Test for Determination of Average Pore Size and Equivalent Opening Size of Filter Fabrics (As Recommended by AASHTO-AGC-ARBA Joint Task Force 17) AASHTO Designation: T______

1. Scope

1.1 This method of test covers a procedure for determining the average pore size and equivalent opening size of filter fabrics both woven and non-woven. The procedure is intended to establish the average pore size and equivalent opening size (EOS) of a filter fabric by a standard sieving technique that uses closely graded spherical glass beads.

2. Apparatus and Supplies

2.1 Soil sieve shaker that meets the shaking requirements of ASTM STP-447 Manual on Test Sieving Methods.

2.2 Sieve pan and cover (8-inch diameter).

2.3 Sieve frame without screen (8-inch diameter).

2.4 Closely graded spherical glass beads (98, within specific range)1 from each of the following grainsize ranges:

U.S. Standard Mesh	Diameter - mm	Corresponding EOS
25-30	0.710 - 0.600	30
35-40	0.500 - 0.420	40
45-50	0.355 - 0.300	50
60-70	0.250 - 0.-212	70
80-100	0.177 - 0.149	100

2.5 Mass balance (Accurate to ± 0.01).

2.6 Five samples (10-inch diameter) of the test fabric.

Supplier: Cataphote Div., Ferro Corp., P.O. Box 2369 Jackson, MS 39205; Telephone (601)939-4631

---

3. Procedure

3.1 Weigh empty sieve pan: record on data sheet.

3.2 Position a fabric sample between sieve frame and pan. Fabric should be attached to the sieve in such a way that no beads can pass between the fabric and the sieve wall.

3.3 Place 50 g of glass beads in the size range of 80-100 U.S. Standard Mesh on top of fabric.

3.4 Place cover on sieve frame.

3.5 Agitate frame and pan in sieve shaker for 10 min.

3.6 Weigh sieve pan and any beads that have fallen into it record on data sheet.

3.7 Calculate weight of beads that passed through fabric (subtract weight of empty pan from weight of pan and beads); record on data sheet.

3.8 Calculate percent of beads retained by fabric according to Equation 1; record on data sheet.

Equation 1

$$\%\text{ beads retained} = \frac{50\text{ g} - \text{weight of beads passing}}{50\text{g}}$$

3.9 Remove residual beads from within and on top of fabric (by light shaking and thumping four to five times with finger).

3.10 Repeat above procedure for each size range of glass beads, working from fine to coarse bead sizes.

3.11 When all size ranges of beads have been tested, plot the percent of beads retained (percent finer pores) as a function of the minimum bead (pore) size on semilog paper to generate a bead size retention distribution curve (Figure 1 - sample graph paper).

3.12 Repeat steps 1-11 four times using a new sample of the test fabric for each trial.

---

4. Calculations

4.1 From the bead size retention distribution diagrams for each of the five trials, read and record the pore size at which 95% of the beads are retained (P953 and that at which 5% are retained (P5). Pore values for a fabric are defined as the mathematical average of all 5 test repetitions for the fabric.

4.2 Calculate the average pore size (Pav9) of the fabric according to Equation 2.

Equation 2

$$P_{avg} = \frac{P_{95} + P_5}{2}$$

For most filter fabrics, average pore size can be approximated visually from the bead size retention distribution diagram because the curve has a near vertical slope around P_{avg}.

4.3 The equivalent opening size (EOS) of the fabric sample is the "retained on" U.S. Standard Sieve number, of that size of beads of which five percent or less by weight passes through the fabric. (See Section 2.4)

E.2 Proposed Standard Specification for Plastic Filter Cloth for Sub-Surface Drainage (As Recommended by AASHTO-AGC-ARBA Joint Task Force 17)

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1. Scope

1.1 This specification covers plastic filter cloth for use in sub-surface drainage.

---

2. Basis of Purchase

2.1 All plastic filter cloth covered by this specification is ordered to an equivalent opening size (EOS).

2.2 Orders for material to this specification shall include the following information, as necessary, to adequately describe the desired product.

2.2.1 Name of material

2.2.2 Equivalent opening size

2.2.3 AASHTO designation number and date of issue

2.2.4 Dimensions (standard width and length)

2.3 Typical ordering descriptions are as follows:

2.3.1 (Trade Name) EOS 70 AASHTO 18 feet wide by 300 feet long

---

3. Cloth

The plastic yarn shall consist of any long-chain synthetic polymer and shall be resistant to deterioration due to ultraviolet and/or heat exposure through basic formulation or the addition of stabilizers and/or inhibitors. The cloth should be calendered or otherwise finished so that yarns will retain their relative position with respect to each other. The edges of the cloth shall be salvaged or otherwise finished to prevent the outer yarn or fibers from pulling away from the cloth, if required.

---

4. Tests and Certification by Manufacturer

4.1 The manufacturer shall make adequate tests and measurements to insure that the material produced complies with this specification.

4.2 The manufacturer shall furnish a mill certificate or affidavit signed by a company officer authorized to sign such documents. The mill certificate or affidavit shall attest that the cloth meets the requirements stated in these specifications.

---

5. Sampling by Purchaser

5.1 The purchaser may make random checks to assure compliance with this specification.

5.2 Samples of plastic filter cloth shall be collected by the manufacturer from each lot of material selected at random in accordance with the sampling plan as specified by the purchaser. Each sample shall carry an identification number to relate it to the stock materials.

5.3 Materials tested by the purchaser and found not conforming to these specifications will be subject to rejection.

---

6. Test for Plastic Filter Cloth

6.1 Determination of Equivalent Opening Size (EOS)

Five unaged samples shall be tested. Obtain 50 gm of each of the following fractions of standard glass beads.

U.S. Standard Sieve Number

Designated EOS	Passing	Retained On
30	25	30
40	35	40
50	45	50
70	60	70
100	80	100

The cloth shall be affixed to a standard sieve having openings larger than the coarsest sand or beads used in such a manner that no sand or beads can pass between the cloth and the sieve wall. The sand shall be oven dried. Shaking shall be continued for 20 minutes. Determine by sieving (using successively coarser fractions) that fraction of beads of which 5 percent or less by weight passes the cloth, the equivalent opening size of the cloth sample is the "retained on" U.S. Standard Sieve number of this fraction. The above shall be run in accordance with AASHTO T-_____(Refer to Appendix E-1).

NOTE: Acceptable filter fabrics normally fall between an EOS of 70 to 100.

6.2 Water Permeability Test

A permeability test shall be run with chosen soil in accordance with AASHTO T215 to determine permeability of the soil alone. A second permeability test shall be run with the same chosen soil at the same compaction according to AASHTO T215 <u>modified</u> to include placement of the sample filter cloth between the soil and the porous disk or screen on the outflow (bottom) end of the permeameter. The filter cloth shall be sealed to the cylinder walls (inside permeameter) to force all flow through the filter fabric. <u>The filter fabric shall not decrease the permeability of the soil</u> (determined in test with soil alone) <u>by more than 10 percent</u> unless approved by the engineer.

When tested against a standard sand, if the introduction of the filter fabric reduces the permeability more than 20%. that the test be repeated using the in situ soil prior to acceptance of the fabric for that project. If, however, the intent is to determine the maximum limiting permeability of the fabric, it may be desirable to use a relatively high permeability/granular soil. Such a test should indicate a fabric's potential for restricting flow from a highly permeable soil. This test will not predict fabric performance in the field, however, it should provide a basis for judging a fabric's permeability performance with granular/noncohesive soils of a lesser permeability. The highly permeable granular soil may be standardized as a non-plastic soil with a D_{10} particle. size between 50 and 125 microns (see attached GSD) and an approximate "k" = 10^{-3} cm/sec at 0% relative density (uncompacted).

AASHTO T215 does not specify the compaction level of the soil used in the permeability test. There are suggestions for Minimum Density (0% Relative Density - placed loose), Maximum Density (100% Relative Density - compacted until soil shows "no visible motion of surface particles adjacent to the edges of the tamping foot"), and Relative Density Intermediate between 0 and 100% (see AASHTO T215 -section 5.5).

The highest soil "k" values in this permeability test will result from a minimum soil density (loosely placed with no compactive effort), thus yielding conservative test results. Therefore, 0% relative density is recommended for the test setup.

AASHTO T215 calls for increasing ΔH until both laminar and turbulent flow ranges have been established from a plot-of system flow velocity versus hydraulic gradient. (section 6.2)

To expedite the evaluation determine the system "k" at hydraulic gradient (i) of 0.5 and 1.0. These gradients should be within the laminar flow range of the system (i.e., k should be the same for all "i" values). These gradients are also representative or conservative for most subsurface drainage conditions. Measure "V" and "i" three times successively for each hydraulic gradient run. Calculate "k" = V / i. Refer to AASHTO T215.

6.3 Puncture Strength

Five samples, unaged, shall be tested using method ASTM D 751, and the puncture strength determined using the Tension Testing Machine with ring clamp, except that the steel ball shall be replaced with a 5/16-inch diameter solid steel cylinder with squared edge centered within the ring clamp. The strengths shall be not less than 40 pounds.

6.4 Breaking Load and Elongation Test

Five warp and five fill samples, unaged, shall be tested in accordance with Method ASTM D1682 using the Wet Grab Test Method. The jaws shall be 1 inch square and the constant rate of travel 12 inches per minute. The strength both parallel to and perpendicular to the direction of the principal yarn shall be not less than 100 pounds.   The elongation at failure shall be at least 10 percent. The product of strength in pounds, times percent elongation shall not be less than 6000.

6.5 Alkali Test

Five warp and five fill samples, 4 ± 0.2 inches by 6 ± 0.2 inches, unages, shall be submerged in a one-liter glass beaker filled to within two inches of its top with a solution of equal amounts of chemically pure sodium hydroxide and potassium hydroxide dissolved in about a liter of distilled water so as to obtain a pH of 11 ± 0.1. The beaker shall be covered with a watch glass and placed in a constant temperature bath at 145 ± 5° F. Using a 1/4-inch I.D. glass tube inserted into the spouted beaker to within 1/2-inch of the beaker bottom, air shall be bubbled gently through the solution at the rate of one bubble per second continuously for 14 days. The solution shall be changed every 24 hours, the new solution being warmed to 150 ± 1° F before replacing the old solution. Each sample then shall be tested for tensile strength and elongation using method ASTM D1862 as described in section 6.4. The strength in either principal direction shall be less than 90 percent of the strength of unaged samples. The elongation at failure shall not be less than 10 percent.

6.6 Acid Test

The test shall be performed as described in section 6.5, except that the solution shall be made of sufficient hydrochloric acid in one liter of distilled water to produce a pH of 3 +0.1. Strength and elongation requirements shall be the same as specified in section 6.5.

6.7 Low Temperature Test

Five warp and five fill samples 4 ± 0.2 inches by 6 ± 0.2 inches, unaged, shall be placed in a refrigerator at 0 ± 3°F for 48 ± 2 hours, then each sample shall be tested at the test temperature using method ASTM D 1682 as described in section 6.4. The strength in either principal direction shall not be less than 85 percent the strength of unaged samples. Elongation at failure shall be at least 10 percent.

6.8 High Temperature Test

The test shall be performed as described in section 6.7 except that instead of freezing the samples they shall be placed in a forced draft oven at 120 ± 3°F for 48 ± 2 hours before the strength and elongation tests. The strength in either principal direction shall not be less than 80 percent the strength of unaged samples. The elongation at failure shall be at least 10 percent.

6.9 Freeze-Thaw Test

Five warp and five fill samples 4 ± 0.2 inches by 6 ± 0.2 inches, unaged, shall be subjected to 300 freeze-thaw cycles in which the specimen shall be cooled to -0°F in a freezer, then immersed immediately in water at room temperature. Each cycle shall be 2 hours ± 4 minutes duration. Samples then shall be tested using method ASTM D 1682 as described in section 6.4. The strength in either principal direction shall not be less than 90 percent the strength of unaged samples. The elongation at failure shall be at least 10 percent.

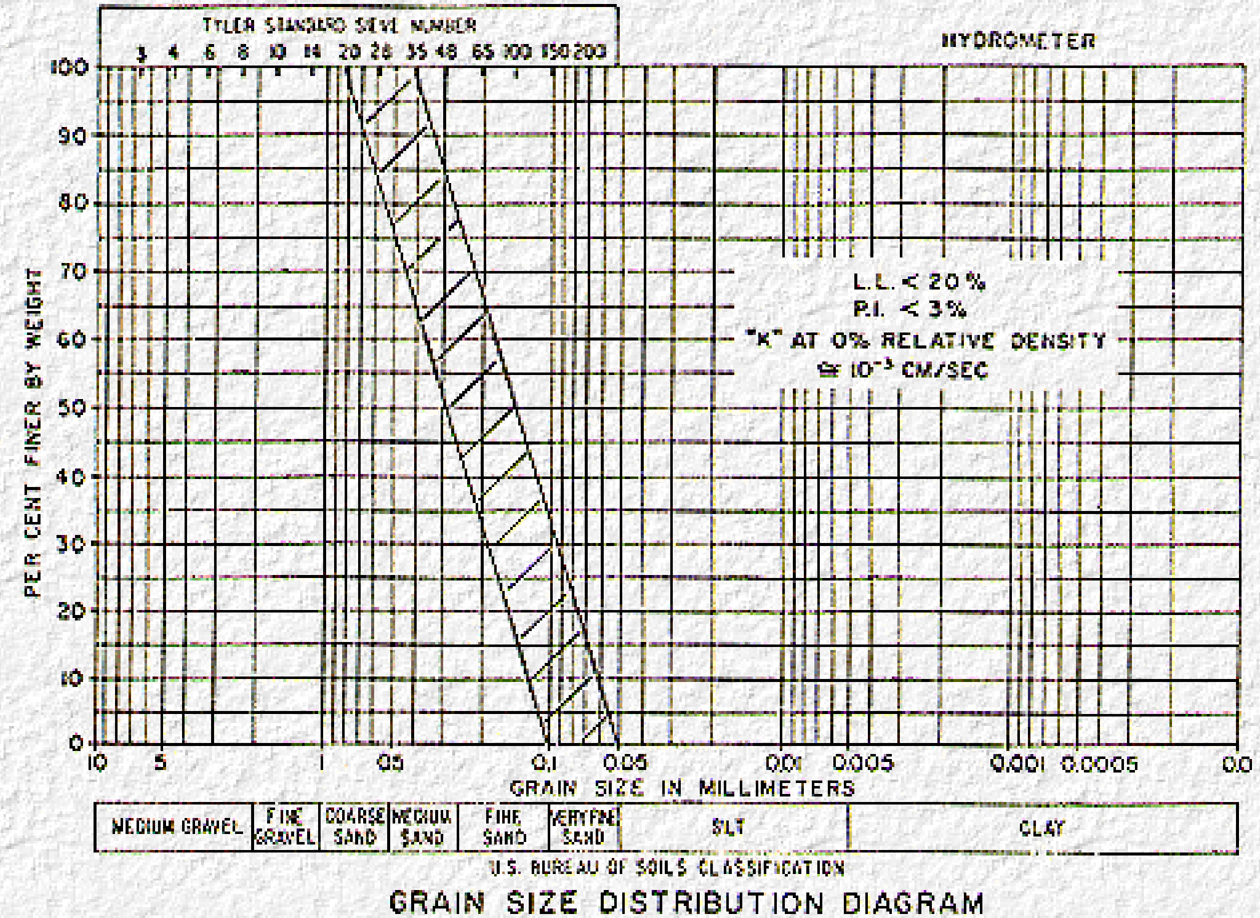
6.10 Weatherometer Test

Five warp and five fill samples 4 ± 0.2 inches by 6 ± 0.2 inches, unaged, shall be tested using Type D or DH (with humidifier off) equipment described in ASTM G 23. Samples shall be exposed continuously for 150 cycles. If fabric is packaged in opaque weather resistant wrap, the test cycles may be reduced to 40. One cycle shall consist of 102 minutes of light only followed by 18 minutes of light with water spray. The black panel temperature, except when the spray is on, shall be 145 ± 9°F. Samples then shall be tested using method ASTM D 1682 as described in section 6.4. The strength in either principal direction shall be not less than 65 percent the strength of unaged samples. The elongation at failure shall be at least 10 percent.

6.11 Bursting Strength

Five samples, unaged, shall be tested in accordance with method ASTM D 751 and the bursting strength determined using the Diaphragm Bursting Tester. The strengths shall be not less than 120 pounds per square inch gage.

Note 1: Tests 6.1 through 6.4 are suggested as quality control tests. Tests 6.5 through 6.11 are suggested as qualifying tests. However, the purchaser may use any of the tests listed to verify compliance.



Recommended Standard Soil for AASHTO Water Permeability Test on Filter Cloth

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Introduction

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The rapid growth of urban areas over the past few decades created the need for construction of extensive storm drainage facilities. Runoff collected by the proliferating paved streets and gutters was collected by storm sewer systems and conveyed directly to the nearest practical disposal point. Over the years, however, it has become apparent that the customary exclusive reliance on storm sewers for surface water disposal creates a series of new problems (1). Among the most critical of these are the following:

. *high peak flows* in storm sewers and streams which require larger facilities at higher costs;

b. *lowering of water tables*, with a detrimental effect on existing vegetation; or salt water intrusion in coastal areas;

c. *reduction in base flows* in receiving streams, affecting aquatic life,

d. *excessive erosion* of streams and sedimentation in lakes, due to higher discharge velocities;

e. *increased pollution* of receiving streams and lakes due to industrial fallout on roofs, fertilizers from lawns and debris from streets and paved areas being conveyed directly to the streams;

f. *Aggravated damage from flooding* due to steadily increasing amounts of runoff.

Nature intended that this water soak back into the earth although present practice prevents it from doing so. In many places the water table has dropped sharply because of insufficient recharging of the ground whereas extensive flooding occurs downstream on a more and more frequent basis. It is obvious that if we continue in this manner, problems will increase to the point where we will be faced with costly damage of great magnitude. The obvious approach would be to design the storm drainage systems that will facilitate nature's process; that is, direct the storm water back into the soil.

New concepts of storm water drainage have developed in recent years. One such concept for disposal of storm water is through use of underground disposal by infiltration drainage. Although this method has not been extensively employed, water resources planners and drainage design engineers are now beginning to consider the infiltration drainage alternative because of the compelling advantages it affords.

The major advantages of using an infiltration system for subsurface disposal of storm water runoff include: 1) the replenishment of groundwater reserves where supplies are being depleted or where overdraft is causing contamination by salt-water intrusion; 2) an economical means of disposing of runoff where conventional methods may require the use of pumping stations or long mains to reach a suitable discharge location; 3) reduction in flow rates by infiltration and storage where the existing outfall is inadequate to carry peak discharges; and 4) a potential for removing pollutants by passage of water through soil. Other benefits include lower costs for surface drainage systems and a reduction in land subsidence. Surface retention prior to infiltration also allows for oxidation of organics and BOD reduction in storm water.

An infiltration drainage system may consist of one or several types of installations. It may be used alone or in combination with conventional systems; serve partially as a detention system and partially as a disposal system. It may be comprised of an open basin; covered disposal trenches utilizing coarse aggregate or pipe with slotted or round perforations; shallow or deep wells; or other components designed to infiltrate the maximum possible volume of runoff into the soil.

The infiltration drainage concept can be incorporated into the design of a transportation facility, commercial development, or subdivision area in many different ways. In the case of the former, little or no additional right-of-way may be required. Side ditches, median areas, unused space within interchanges, small land-locked areas, borrow areas, and space around rest areas are all potential sites. With imaginative planning, infiltration facilities such as the open basins can be terraced and landscaped to offer scenic enhancement and, in some cases, a park-like atmosphere. These systems produce many benefits and cause no negative effects when properly blended with the environment.

Infiltration drainage methods have been used in coastal areas of the United States for groundwater recharge and to solve special drainage problems. They are not limited to coastal areas, however, but may be used in any location where suitable soil conditions exist. Infiltration methods have been used extensively on Long Island, in Florida, parts of Texas, and in California. Research studies by the New York Department of Transportation on recharge basins for highway storm drainage have demonstrated the practicality of the method and, through full scale testing, have validated the design theory. A research study by the California Department of Transportation evaluated infiltration methods in northern California and identified important design considerations as related to highways. Considerable success has been gained in SouthernFlorida with recharge concepts using infiltration trenches. Detention-infiltration systems have also been constructed in Canada. These types of systems may have application in other areas.

This manual has been developed based on experience which was derived from engineering judgment and applied theory. Its purpose is to provide the information necessary to evaluate for feasibility, as well as to plan and design, surface and subsurface infiltration systems or combination systems that can be incorporated into the overall drainage scheme of a particular transportation facility, street system, or commercial development. Basic criteria are presented with examples cited to assist the designer in selecting an appropriate system.

The next two chapters provide introductory and background information on the state-of-the-art utilization of systems for underground disposal of storm water. They provide solutions to problems of groundwater recharge, storm water disposal, and/or prevention of salt-water intrusion. Chapter 3, entitled "General Considerations", includes criteria for the evaluation of alternative disposal systems, environmental and legal considerations, and general guidelines for soils exploration and investigation. Chapter 4 includes specific design guidelines to enable the designer to plan and develop economical and environmentally feasible designs based on local hydrology and soil infiltration characteristics. Numerous design examples are used to aid the reader.

Chapter 5, "Construction Methods and Precautions", and Chapter 6, "Maintenance and Inspection", provide information on the installation and long term performance of various infiltration systems.

The word *infiltration*, is a general term used throughout this manual to describe the flow of water into the soil. In the discussion of trench systems for subsurface disposal of storm water, the term *exfiltration* is used to describe the process in which water flows out of the trench or pipe conduit and into the soil.

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